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Hydrologic Studies in Support of Project Functions

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Angel Fire, New Mexico
August 7-9, 1990

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Seminar Proceedings No.

Proceedings of a Hydrology and Hydraulics Workshop

on

Hydrologic Studies in Support of Project Functions

Angel Fire, New Mexico

August 7-9, 1990

U.S. Army Corps of Engineers
Water Resources Support Center
Hydrologic Engineering Center
609 Second Street
Davis, California 95616
(916)756-1104

FOREWORD

A three-day workshop entitled "Hydrologic Studies in Support of Project Functions" was held in Angel Fire, New Mexico during August 1990. The purpose of the workshop was to provide an informal forum for Corps of Engineers personnel who are routinely involved with hydrologic engineering work to discuss specific issues and exchange ideas related to hydrologic aspects of Corps project functions. The 33 workshop participants represented 21 Corps offices including HQUSACE, division, district and laboratories.

Topics addressed during the workshop and included in these proceedings include four papers on "River and Reservoir Regulation Applications" (Session I), five papers on "Conservation Storage Analysis" (Session II), seven papers on "Advanced Computer Techniques" (Session III), and eight papers on "Operational Hydrology" (Session IV). The papers for each session are preceded by an executive summary of that session. Each paper is followed by a record of the discussion associated with that paper, if any. In Session IV, the recorder included the discussion in the executive summary.

The workshop was co-sponsored by the Hydrologic Engineering Center and the Corps' Committee on Hydrology. The workshop proceedings, in addition to the general seminar planning and coordination, was organized by Mr. R.G. Willey of the Hydrologic Engineering Center. Valuable assistance was graciously provided for chairmanship of the individual sessions by Mr. Dennis Williams, Nashville District; Mr. Gary Dyhouse, St. Louis District; Mr. Loren Pope, Little Rock District; and Mr. Roy Huffman, HQUSACE. Session discussion recorders included the first three chairmen listed above and Mr. Bruce Beach of the Albuquerque District. The general meeting room, the block of individual hotel rooms, the free-time social activities, and the many other necessary local arrangements were handled in an expert and efficient manner by Mr. David Gregory of the Albuquerque District.

The views and conclusions expressed in these proceedings are those of the authors and are not intended to modify or replace official guidance or directives such as engineering regulations, manuals, circulars or technical letters issued by HQUSACE.

R.G. Willey
Editor

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SEMINAR ATTENDEES

HYDROLOGIC STUDIES IN SUPPORT OF PROJECT FUNCTIONS

7-9 August 1990
Angel Fire, New Mexico

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SESSION I
RIVER & RESERVOIR REGULATION APPLICATIONS

SUMMARY OF SESSION I
RIVER AND RESERVOIR REGULATION APPLICATIONS

prepared by

Bruce C. Beach
Albuquerque District

OVERVIEW

The topics covered in the presentations included reservoir system analysis in support of planning or reregulation studies for multipurpose or single purpose navigation or flood control projects.

PAPER PRESENTATIONS

Clinton E. Word, Tulsa District, presented a paper entitled "Arkansas River - Reservoir System Studies." Mr. Word's paper described the use of the reservoir system model "SUPER" to evaluate the effects of proposed new projects, the modification of existing projects, and reregulation on a watershed that has 48 federally-controlled reservoirs. The model used 47 years of daily flows to generate frequency and damage curves for each recommended change to the system. The study was conducted as part of the Arkansas River Basin, Arkansas and Oklahoma, Feasibility Study.

Ronald L. Hula, Southwestern Division, presented a paper entitled "Regulated Flow Peak Discharge Frequency Estimates For Large Basins." Many of the major drainage basins in the Southwestern Division have been modeled using the Southwestern Division Reservoir Regulation Simulation Model. The model uses continuous simulation to generate daily flow values. Peak flow values can be generated by comparing peak flow values to daily flow values for uncontrolled basins. A ratio of peak flow to daily values was generated using the SWD Watershed Model. Verification of results indicate that the procedure increases the accuracy of flood damage computations.

Russell P. Yaworsky, Sacramento District, presented a paper entitled "Reevaluation of Frequency of Regulated Flows on the American River At Sacramento." A reevaluation of Folsom Dam and the American River levees indicates that only a 63-year level of protection is provided. An analysis of unregulated flows was performed to develop volume-frequency relationships. Balanced hydrographs were then created, patterned after the PMF hydrograph. Results were used for plan formulation.

Lyndon C. Richardson, Jr., Ohio River Division, presented a paper entitled "Flow Regulation Model for the Proposed Hinged Pool Operation, Olmstead Locks and Dam, Ohio River." An unsteady flow regulation model was used to provide a hinged pool operation plan for the proposed project. The higher degree of sophistication than the stair step operation now in use is necessary due to the constraint posed by the presence of Paducah, Kentucky, 30 miles upstream. [illegible]e of the model will allow for minimumization of locking time, minimizing wicket gate operation, and reducing surges in the upper and lower pools.

ARKANSAS RIVER - RESERVOIR SYSTEM STUDIES

by

Clinton E. Word[1]

INTRODUCTION

A reservoir system model evaluation was conducted as part of the Arkansas River Basin, Arkansas and Oklahoma, Feasibility Study. The model study focused on the opportunities for new multi-purpose projects, increased flood storage in the existing projects, and improvements to the existing reservoir system operating plan.

A major problem in any system study is the evaluation of the effects of change on other system purposes. This problem is magnified in the Arkansas River where 48 federally-constructed reservoirs are operated for flood control, hydropower, water supply, water quality, sediment control, navigation, recreation, and fish and wildlife. Seventeen of the projects are locks and dams constructed to provide navigation from the mouth of the Arkansas River to the Port of Catoosa near Tulsa, Oklahoma. In addition to these reservoirs, the Grand River Dam Authority, an Oklahoma State agency, has constructed two projects in the Lower Grand River Basin for hydroelectric power and flood control.

The model study was conducted by the Tulsa District Hydrologic Modeling Center using the Southwestern Division Reservoir Regulation Computer Model (commonly referred to as SUPER) for evaluating both the hydrologic and economic impacts. The Arkansas River SUPER model uses 47 years of historical record with a routing interval of one day. Simulations were conducted for each recommended change to the system by making modifications to the model description and allowing the SUPER model to iterate sequentially through each day of the period of record. The simulations determined releases which adhered to the plan of regulation, taking into account hydrologic conditions on each particular day. The results of the simulations were the daily hydrologic conditions that would exist if the 47 years of record were to occur with the described reservoirs and operating scheme. The modified hydrology was then processed with the SUPER Analysis model giving frequency/duration curves and economic damages for each reservoir and river reach.

Each simulation was evaluated by viewing elevation duration/frequency curves for each reservoir, flow duration/frequency curves at each of the 50 control points below the projects, hydropower output, water supply deficiencies and economic damages (agricultural, structural, dredging costs, navigation delay, environmental).

FEASIBILITY STUDY

The Arkansas River Basin, Arkansas and Oklahoma, Feasibility Study was the first cost-shared feasibility study with multiple State sponsors and multiple Corps of Engineers districts participating. State representatives for the non-federal sponsors were the Arkansas Soil and Water Conservation Commission (ASWCC) and the Oklahoma Water Resources Board (OWRB). The Corps of Engineers participants were the Little Rock and Tulsa Districts.

---

[1]Chief Modeling and System Section, Hydraulics and Hydrology Branch, Tulsa District Corps of Engineers.

The principal study partners were involved in the Management of the Study through two committees:

1. The Executive Committee which was chaired by the Tulsa District Engineer and included the Little Rock District Engineer and the directors of the OWRB and the ASWCC.

2. The Study Management Team which was charged with the execution of the study activities and objectives established by the Executive Committee. The Study Management Team was chaired by a representative of the Little Rock District Planning Division. Chief planners from the OWRB, the ASWCC, and Tulsa District also served on this team.

The study was also coordinated with U.S. Fish and Wildlife Service, Institute of Water Resources and the Southwestern Power Administration.

The purpose of the Arkansas River Basin study, which began in March 1984, was to evaluate the need and opportunities for reducing flood damages and for developing additional municipal, industrial, and agricultural water supplies in the Arkansas River Basin in Arkansas and Oklahoma. During the reconnaissance phase of the study, the objectives were expanded to evaluate the potential for improvements to the existing McClellan-Kerr Arkansas River Navigation System. It was recommended that feasibility level studies be conducted that would examine in more detail solutions to navigation, flood control, hydropower, recreation, water supply, and fish and wildlife problems within the basin in the two states.

There were two principal measures to address the problems and opportunities: to increase the available storage in the basin through modification of existing projects or construction of new projects, and to modify the system operating plan to achieve a reasonable balance of purposes for which the projects are operated.

DESCRIPTION OF BASIN AND EXISTING PROJECTS

The Arkansas River begins on the eastern face of the Rocky Mountains near Leadville, Colorado, and flows southeasterly nearly 1,400 miles through Colorado, Kansas, Oklahoma, and Arkansas to join the Mississippi River. The basin comprises about 138,000 square miles of contributing drainage area with about 128,000 square miles above Van Buren, Arkansas (Oklahoma and Arkansas state line.) The Arkansas River system currently consists of 48 federally-constructed reservoirs operated for flood control, hydropower, water supply, water quality, sediment control, navigation, recreation, and fish and wildlife. Seventeen of the 48 projects in the Arkansas River system are locks and dams constructed to provide navigation from the mouth of the Arkansas River to the Port of Catoosa near Tulsa, Oklahoma. In addition to these reservoirs, the Grand River Dam Authority, an Oklahoma State agency, has constructed two projects in the Lower Grand River Basin for hydroelectric power and flood control. A map of the Arkansas River Basin is shown on Figure 1.

Flood Control. Flows on the main stem of the Arkansas River are modified primarily by 11 Oklahoma storage projects which provide about 7.7 million acre-feet of flood control storage. That storage represents in excess of 70 percent of the total flood control storage in the basin. The 11 projects are listed in Table 1. Runoff on about 7,500 square miles of drainage area below the 11 projects and above Van Buren, Arkansas, is uncontrolled.

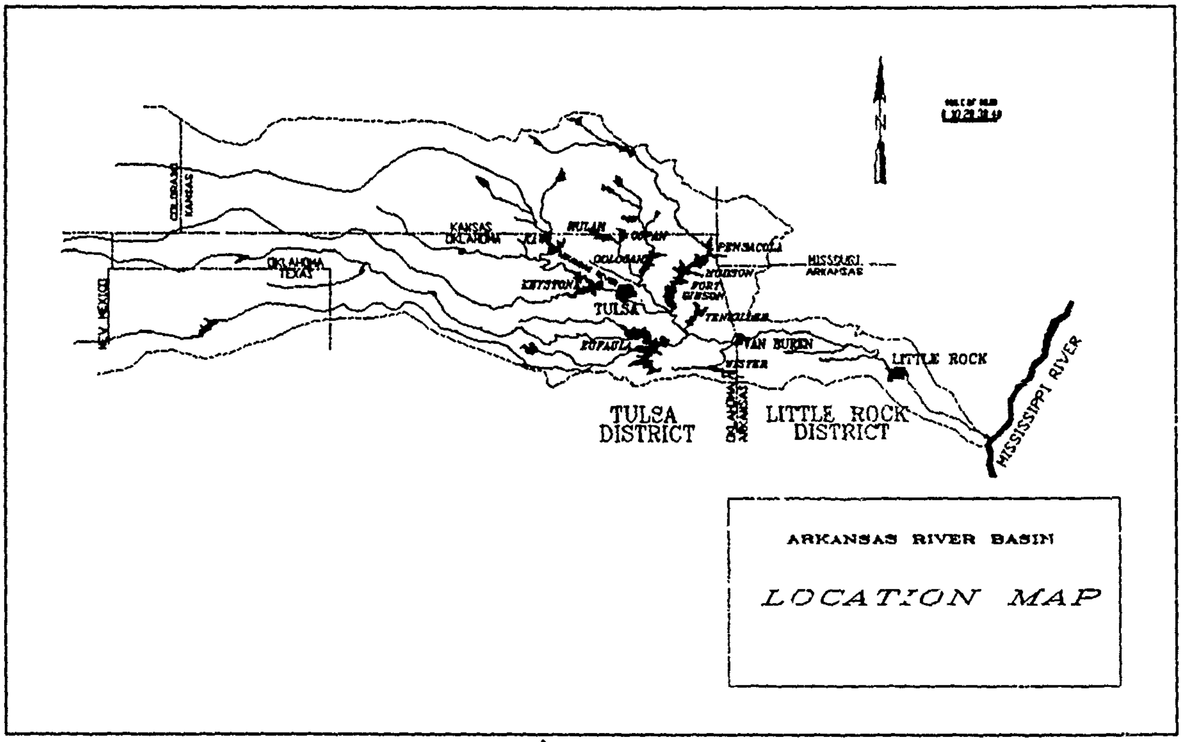


Figure 1

The primary objective of a system water control plan is to achieve a reasonable balance of purposes for which the projects are operated. The Fort Smith/Van Buren, Arkansas, area near the Oklahoma-Arkansas state line is the primary control point for the lower Arkansas River navigation system (Van Buren to the mouth). The 11 principal upstream storage projects are operated to maintain flow targets at the Van Buren gage and all of the reservoir releases flow past this point. Seasonal guide curves have been developed and analyzed for system operation. These guide curves relate the flow at the Van Buren river gage with the percent of flood control storage utilized in the 11 upstream multi-purpose storage projects.

Figure 2 shows a Van Buren guide curve that is representative of the current system operation. The objectives of this plan are to increase the number of days below 75,000 c.f.s. on the navigation system, to provide a taper from flood control releases to conservation operation, and have minimal impacts on hydropower, recreation and flood control. The system has been operated under this plan since June 1986.

Navigation. In 1989, the McClellan-Kerr Waterway transported an estimated 8.4 million tons of commodities and has become an important segment of the region's transportation network. High flow conditions increase fuel, labor, and capital costs due to the increased time required for movements, reduced tow sizes, and increased accident rates. Recession of high-flow events also cause periodic delays and blockages due to shoaling which adds to total transportation costs. Therefore, an important phase of this study involved examining alternative plans that would enhance the navigation potential of the system.

TABLE 1

ELEVEN PRINCIPAL UPSTREAM STORAGE RESERVOIRS
IN THE ARKANSAS RIVER BASIN

Project	River	Flood Control Storage (acre-feet)
Keystone	Arkansas	1,180,000
Oologah	Verdigris	965,600
Pensacola	Grand (Neosho)	525,000
Hudson	Grand (Neosho)	244,200
Fort Gibson	Grand (Neosho)	919,200
Tenkiller	Illinois	576,700
Eufaula	Canadian	1,510,800
Kaw	Arkansas	919,400
Hulah	Caney	257,900
Copan	Little Caney	184,300
Wister	Poteau	386,800
Total		7,669,900

Recreation. Recreation facilities located in the basin, both around the reservoirs and in parks and recreation areas along the main stem of the Arkansas River, are an important resource. Visitor-day occasions at the Oklahoma reservoirs have averaged over 21 million in recent years, while the number of activity occasions experienced in the parks and recreation areas along the main stem of the river have averaged about 14 million annually.

Hydropower. Installed generating capacity at the reservoir sites and at the run-of-the-river plants totaled 680,000 kilowatts in 1988. These plants produced an estimated 3 million megawatt-hours (mWh) of electricity valued at $90,000,000.

SOUTHWESTERN DIVISION RESERVOIR REGULATION COMPUTER MODEL

The Southwestern Division Reservoir Regulation Computer Model (commonly referred to as SUPER) is a tool for evaluating the hydrologic and economic impacts of a given plan on a multi-purpose system of reservoirs.

The SUPER model is a period of record simulation model using a routing interval of one day. The hydrologic input to the model, for every reservoir and stream control point, is the period of record uncontrolled area flow. The development of these uncontrolled area hydrographs is based on computations which utilize all available pertinent daily records and multi-reach storage vs. discharge (Puls) stream routing relationships.

The basic input data required to describe the reservoirs includes area-capacity curves and maximum and minimum discharge curves. The relationship of the reservoirs is defined by a seasonal function of storage vs. level for each reservoir. Two reservoirs are considered in balance when they are at the same level as determined from their respective storage-level functions and contents. The relationship of each reservoir to other reservoirs and stream control points is provided by a set of Muskingum routing coefficients for each control point below the reservoir. The regulating discharge criteria for all stream control points is supplied as a seasonal function of a system state parameter (reservoir level or system percent full.)

The SUPER model iterates sequentially through each day of the period of record determining releases which adhere to the plan of regulation taking into account current and forecasted hydrologic conditions on each particular day. Each iteration includes: simulated reservoir and control point flow forecast, mandatory flood control releases required if the flood control pool would be exceeded during the forecast period, mandatory releases for water quality, water supply or hydropower, regulating discharge for each day of the forecast period based on the appropriate parameter for each control point, and a flood control release schedule such that releases do not exceed the available space at downstream control points.

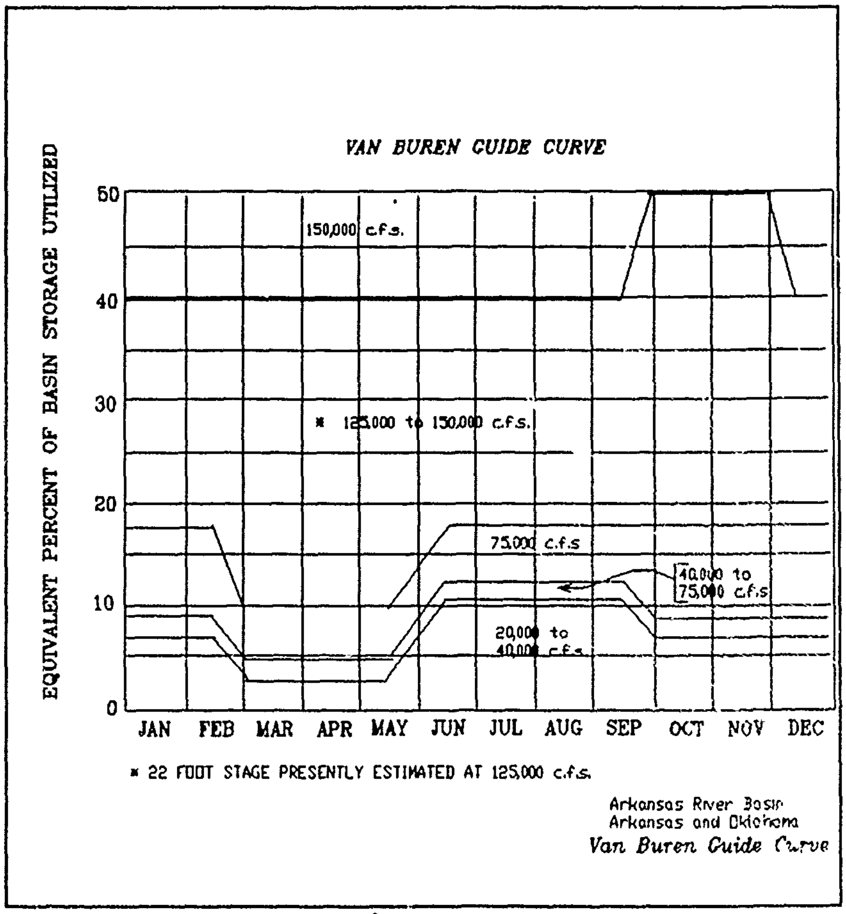


Figure 2

STUDY

The major problem in this study was not the selection of new project sites or operation plans. Every interest group had multiple suggestions for "improvements" to the system. The major problem was to evaluate the effects of the proposed changes on the other system purposes. These proposed changes included 11 new multi-purpose projects, modification to present storage projects and 18 modifications of the existing plan of regulation. The SUPER model was used to evaluate alternative plans on other system's purposes including navigation, flood control, hydropower generation, recreation, and environmental and cultural resources.

The remainder of this paper is a presentation of three sample plans studied and the methodology of analysis for each project purpose. It is noted that the evaluations of impacts were performed in two manners and except for the magnitude were similar in terms of the relative differences between plans. The feasibility report analyses of economic impacts were computed in a traditional manner external to the SUPER model, utilizing hydrologic output from the regulation simulation model. Average annual benefits were computed by noting the differences between operating plans. Initial analysis used by a water management study group to evaluate the impacts of alternative operating plans were performed using average yearly economic outputs of the SUPER models for the 47-year period of record.

Each plan was simulated by modifying the existing Arkansas River Basin SUPER model and allowing the computer to simulate the hydrologic effects of 47 years of record on the new system. The results were then evaluated by comparing the duration and frequency curves for the existing projects and all downstream control points, the hydropower produced and damages to agriculture, structures, navigation, recreation, environmental and cultural resources.

The current operating plan (known in the Feasibility Repor Plan C) and the operating plan that was utilized from 1979 until the adoption of the current plan in 1986 (Plan B) were simulated as base runs for comparison purposes.

One of the requests made by navigation interest was to maintain a flow of no more than 75,000 cfs at Van Buren, Arkansas for 365 days a year. In an attempt to determine the limits of the existing system to accomplish this a simulation was run to determine the maximum number of days the Van Buren flows could be held below 75,000 cfs using all of the existing upstream flood control storage available. The economic and hydrologic impacts of using 100 percent of the flood control storage in the 11 regulating projects to maintain a maximum of 75,000 cfs at Van Buren were evaluated by viewing the number of days this flow was equaled or exceeded during the period of record, the elevation-duration curves for the 11 projects, and the residual flood damages produced from the SUPER analysis.

The use of 100 percent of the flood control storage resulted in a yearly average of 356 days of flows below 80,000 cfs. (Note: Due to the fact that the simulation held the flow at Van Buren at 75,000 cfs, any additional local flow, no matter how small, would cause the 75,000 cfs count to show exceeded. The count, therefore, was taken at 80,000 cfs.) The number of days below 75,000 cfs equaled or exceeded on a yearly basis ranged from 307 days in 1973, to 365 days in the dry years.

A similar simulation was made to determine the amount of added storage at each of the existing projects that would be required to control the flows at Van Buren to a maximum of 75,000 cfs. The system was run with unlimited flood control storage in the nine major controlling storage projects (Fort Gibson, Oologah, Hulah, Copan, Kaw, Keystone, Tenkiller, Eufaula, and Wister), and the Van Buren guide curve was modified to restrict the flow to 75,000 cfs with the taper as described in Plan C. This run resulted in two answers: maximum storage that could be used for enhancement of navigation and the stream on which this storage should be located.

The amount of equivalent storage at or above these existing projects needed to control the system below 75,000 cfs was approximat 15 million additional acre-feet, and is shown in Table 2. This storage would give an average c 361 days per year below 75,000 cfs at the Van Buren gage. The remainder of the floods are due to storms below the projects and are not controllable with the current system even with increased storage capacity.

A request was also made to evaluate the effect of a power drawdown into the conservation pool. It was anticipated that the power drawdown would help with the taper operation. Power projects were drawn into the conservation pool seasonally by changing the elevation associated with the top of the conservation pool. It was found that this did not significantly increase the success of the taper operation since the drawdown preceded the flood season and most tapers occur during or following flood season. In addition, there were no significant decreases in flood damages and little increase in power generated.

TABLE 2

FLOOD STORAGE REQUIRED
FOR 75,000 CFS AT VAN BUREN
(in acre-feet)

Reservoir	Maximum Storage	Current Existing Storage	Additional Storage Required
Fort Gibson	4,560,101	1,284,400	3,275,701
Oologah	2,881,265	1,519,000	1,362,265
Hulah	445,600	289,088	156,512
Copan	523,818	227,730	296,088
Kaw	193,145	143,000	50,145
Keystone	5,776,390	1,737,631	4,038,759
Tenkiller	2,533,858	1,230,800	1,303,058
Eufaula	7,536,703	3,825,362	3,711,341
Wister	1,013,185	427,900	585,285
Total	25,464,065	10,684,911	14,779,154

Flood control interest requested evaluation of impacts for an accelerated evacuation of the flood storage when the system is nearly 75 percent full. The Van Buren guide curve was changed to allow evacuation of the system at 150,000 cfs or the maximum uncontrolled peak experienced during this event up to 250,000 cfs when the system exceeds 75 percent of flood control storage. The simulation indicated no significant change in pool durations, flood damages, or navigation impacts. This may be due to the fact that additional floods occurring when the system is 75 percent full is an improbable event. The team did however recommend that this feature be incorporated into the final operating plans since it did increase the system flood control capacity without significantly increasing the flood damages.

Multiple simulations were also made to evaluate the impacts of plans which modify the balancing scheme of the 11 upstream projects in an effort to protect projects above major damage centers. It was found that changes in the balancing rule curves did not significantly alter the damages; therefore, there was no recommendation to include a modified balancing curve in the final system operating plans.

Recommended Plan for Additional Evaluation

Based on the evaluation of modifications to the existing operating scheme, an operating plan was recommended by the study group. Following careful consideration of each plan, it was decided to combine attractive features of several plans into one. The preferred alternative (Plan D) has the combined features of a 60,000 cfs bench instead of a 75,000 cfs bench; accelerated evacuation at 150,000 cfs or a maximum of 250,000 cfs above 75 percent full; and, reduced basin storage levels in the fall months. The 75,000 cfs bench was changed to a 60,000 cfs bench so that dredging operations could proceed during the bench thus lessening the impact of high flows.

METHODOLOGY OF EVALUATION

Flood Control. For purposes of this study, flood damages were estimated for two separate categories - damages to crops (agriculture), and damages to structures and contents.

Agricultural. Agricultural losses are based on crop-specific damage functions that incorporate seasonal factors with economic data to generate loss estimates. The crop loss functions were used to calculate losses from actual flood events, and thus vary from year to year depending upon the severity of the flooding experienced. A major determinant of the percent loss in these calculations is the time of year the flood occurs. Crop distributions were verified through consultations with Agricultural Extension personnel and publications for the counties involved. Costs, yields, and commodity prices are based on the latest estimates.

Structures. Average annual estimates of damages to structures and contents in the floodplain areas were estimated by developing elevation-damage functions for each gaging station used in the analysis. Floodplain inventory data were used to establish the base values of properties at risk. This function includes farm buildings, machinery, fences, roads, bridges, and residential and commercial buildings and contents. Stage-damage curves were derived for both rural and urban structures within the various reaches. Expected annual flood damage computations were estimated using the Hydrologic Engineering Center (HEC) Expected Annual Damage (EAD) computer program package. Frequency relationships were obtained from SUPER for each of the plans evaluated.

In-pool damages caused by fluctuations in pool elevations within the reservoirs include federally-owned recreation facilities, State park and recreation areas, and private marinas. An inventory of all existing development was conducted, including the number, type, and elevation of all structures, along with estimated elevation-damage relationships for each reservoir. Table 3 displays summary pool elevation-duration data for selected Oklahoma reservoirs for three operational plans.

TABLE 3

SUMMARY OF POOL ELEVATION - DURATION DATA

Lake	% of Flood Storage	Days Per Year Equaled or Exceeded Plan B	Plan C	Plan D
Tenkiller	0	122	127	125
	24	14	15	13
	50	6	6	5
Eufaula	0	111	114	115
	15	25	27	25
	22	18	18	15
	46	8	8	6
Keystone	0	131	133	135
	22	15	16	12
	50	5	5	4

Navigation. Changes in the operating plans for the McClellan-Kerr Arkansas River Navigation System are manifested directly in the level and duration of flows downstream from the various release points. A flow rate of approximately 60,000 cfs at the Van Buren gage is considered by navigation interests to be a critical level. (Note: The navigation study indicates that a 75,000 cfs flow in the system is the maximum for economical navigation. The flow of 60,000 cfs was chosen at the Van Buren gage because it translates to approximately a 75,000 cfs flow in the lower portions of the system around Little Rock, Arkansas.) Above that rate, tow operators begin to experience significant cost increases due to the use of smaller tows and double tripping, which increase the ton-mile costs of shipping.

Flow rates and durations on the system directly impact fuel and time costs and indirectly affect other navigation costs, including delays caused by shoaling and dredging. Fuel and time cost functions that were used to evaluate plan impacts were developed by Gulf South Research Corporation, and are described in detail in the 1987 Report, Economic Impacts of Alternate Regulation Plans on Navigation on the Arkansas River Navigation System. These functions were adjusted to reflect future levels of traffic on the system. Future tonnages were obtained from the 1988 Inland Waterway Review, published by the U.S. Army Institute for Water Resources.

Delay costs due to shoaling, and related dredging costs were derived from functions that have been developed by the Hydrologic Engineering Section at Southwestern Division and are related to SUPER simulations. Both the delays resulting from shoaling, which follows high flow events, and the subsequent dredging operations represent significant costs to navigation. Therefore, the plan which is most effective in reducing these costs will likely be the preferred plan from the perspective of navigation interests.

Table 4 shows the impacts of the three alternate plans on the critical flow rates at three gages on the river; at Muskogee in Oklahoma, and at Van Buren and Little Rock in Arkansas. These flow data reveal that Plan D produces an average of nine fewer days per year of flows above the critical (60,000 cfs) level at the Van Buren gage, and eight fewer days annually at Muskogee.

Table 5 shows the impacts of the three alternative plans on the cost items described above. The differences brought about by Plan D are positive, whereas the impacts of Plan B are uniformly negative. The relative changes in total costs that result from either plan are minute, with the ratio of benefits to base plan costs being only six-tenths of one percent in the case of Plan D, and even less with Plan B.

Recreation. Both the Little Rock and Tulsa Districts undertook extensive and detailed studies to evaluate the impacts of different operating plans on recreation. The studies involved parks and recreation areas along the main stem of the Arkansas River as well as recreation use on several large reservoirs in the Tulsa District. Recreation impacts are summarized in Table 5.

Six lakes in the Arkansas River system are most sensitive to the competition of purposes. The lakes include Fort Gibson, Tenkiller, Kaw, Keystone, Oologah, and Eufaula. Each of these lakes has over a million recreation visitors per year.

The Tulsa District undertook a detailed and complex effort to estimate the relationships between pool elevation and recreation visitation. These relationships were used to estimate the average amount of visitation losses associated with each of the alternative operating plans. Additionally, from January through March 1989, interviews were conducted with the project management staff of each of the above reservoirs. Two general questions were asked. First,

TABLE 4

DAYS OF FLOW LESS THAN SPECIFIED
DISCHARGES BY PLAN

Gage/Location	Discharge (cfs) 60,000	75,000	90,000
Muskogee			
Base Plan	332	342	349
Plan B	331	341	349
Plan D	333	341	349
Van Buren			
Base Plan	308	326	334
Plan B	310	322	331
Plan D	317	328	334
Little Rock			
Base Plan	285	305	320
Plan B	288	304	317
Plan D	287	309	322

would the differences between the current and alternative operating plans vary to the degree that changes in the quality of recreation can be measured? And second, would changes made under the alternative operating plans result in more recreation facilities being closed than under the current plan?

Hydropower. The impacts of alternative operating plans on the production of hydropower were based on the assumption that changes in the operating plan would not result in a change in marketable capacity from the existing projects and, therefore, capacity benefits would be unchanged.

Hydropower energy production in megawatt hours (MWh) and the resultant energy value in dollars were analyzed for both the upstream Oklahoma reservoirs and the run-of-river low head generating plants at locks and dams. Both the absolute and relative positive impacts of the average annual hydropower generation were far greater than all of the other impacts to the other system purposes combined. This was to be expected, as the positive impact from the generation of hydropower occurs on a daily basis whereas the negative impacts to other system purposes occur seasonally or infrequently as a result of extreme hydrologic conditions. Energy generation estimated by SUPER was within ten percent of recent historical generation, indicating that the input power loading is representative of current conditions. This indicated that this was a reasonable hydropower loading for evaluating the impacts on other purposes. In review of upstream hydropower none of the plans reduce the capacity of the system and in all cases sufficient hydropower storage was available for meeting additional loads if required. Therefore, only the changes in energy from the run-of-river plants were used in comparison of the plans.

TABLE 5

SUMMARY OF OPERATING PLAN BENEFITS
(Average Annual Values in $1,000)

Item	Plan C	Plan B	Plan D
Navigation			
Fuel Costs	3,894	3,895	3,891
Time Costs	9,039	9,048	9,037
Blocked Navigation (Shoaling)	417	432	353
Dredging	1,820	1,852	1,796
Subtotal	15,170	15,227	15,077
Flood Control			
Arkansas			
Agricultural Damages	1,580	1,452	1,553
Other Urban-Rural Damages 1/	435	432	437
Total Flood Damages-Arkansas	2,015	1,884	1,990
Oklahoma			
Agricultural Damages	2,938	2,774	2,858
Other Urban-Rural Damages 1/	10,596	10,479	10,456
In Pool - Oklahoma 2/	1,313	1,291	1,287
Total Flood Damages-Oklahoma	14,847	14,544	14,601
Subtotal	16,862	16,428	16,591
Hydropower 3/5/			
Energy Values			
Reservoir Projects	39,176	39,138	38,933
Locks and Dams	50,434	49,880	50,570
Subtotal	89,610	89,018	89,503
Recreation 4/Visitor-Day Values			
Arkansas	45,533	45,533	45,533
Oklahoma	71,987	71,989	72,019
Total	117,520	117,522	117,551

Notes:

1/ Flood damages to miscellaneous urban and rural property, including roads and bridges.
2/ Flood damages to Federal and State recreation facilities and private marinas.
3/ Average yearly hydropower generation in MWh valued at an energy value of 29 mills/KWh.
4/ Recreation values estimated at $3.20 per visitor day.
5/ None of the plans restricted the system hydropower storage projects from meeting their required loads.

Summary of System Operating Plan Impacts. Table 5 displays the impacts of each of the three final alternative operating plans by project purpose, and summarizes the total effects. The results indicate that, in total, there is little difference in benefits among the three alternatives. This statement is also true for each of the individual project purposes.

CONCLUSION

The use of the Southwestern Division Reservoir Regulation Computer Model (SUPER) made it possible to evaluate both the hydrologic and economic impacts of recommended changes to the Arkansas River system of reservoirs on other system purposes. Without a model study it would be impossible to evaluate the effect of a change on flood control, hydropower, water supply, water quality, sediment control, navigation, recreation, and fish and wildlife. The study was able to establish limits of benefit to modifications to the system as well as answer "what if" type questions for all interested parties. Using the model it is believed that a reasonable balance of purposes has been achieved in the Arkansas River system operating plan.

REFERENCES

Arkansas River Basin, Arkansas and Oklahoma Feasibility Report March 1990

Beard, Leo R., Statistical Methods in Hydrology, January 1962.

Coomes, R. Terry, Regulation of Arkansas Basin Reservoirs, ASCE National Workshop on Reservoir Systems Operations, Boulder, Colorado 1979.

Copley, Ross R., Arkansas River System Regulation Study, ASCE National Workshop on Reservoir Systems Operations, Boulder, Colorado 1979.

Fish and Wildlife Coordination Act Report on the Arkansas River Feasibility Study, Arkansas and Oklahoma, U.S. Fish and Wildlife, Vicksburg, Mississippi.

Fish and Wildlife Coordination Act Report on the Arkansas River Feasibility Study, Arkansas and Oklahoma, U.S. Fish and Wildlife, Tulsa, Oklahoma.

Gulf South Research Corporation, Economic Impacts of Alternate Regulation Plans on Navigation on the Arkansas River Navigation System, 1987.

Hight, Ralph R. and Munsell, Fred J., Case Study of Hydropower Formulation, Kaw Lake OK., Waterpower '83 International conference on Hydropower.

Hula, Ronald L., Southwestern Division Reservoir Regulation Simulation Model, ASCE National Workshop on Reservoir Systems Operations, Boulder, Colorado 1979.

Arkansas River - Reservoir System Studies

by

Clinton E. Word

SUMMARY OF DISCUSSIONS BY BRUCE C. BEACH

In response to a query about the applicability of the SUPER model for forecasting, Mr. Word stated that SUPER is a planning model but that it was used in the last flood for forecasting drawdown times on the recession limb of the flood.

A discussion of the severity of the last two floods ensued, with the author stating that they both were rare events, the 1986 flood varying from 50-year to several hundred and the frequency of the 1990 flows have not been determined but the rainfall was extreme, 50% more that the 100-year value.

In response to a series of questions, the author stated that most of the projects have sediment pools with 50 year design lives. Some projects have low flow augmentation, three for navigation. He also stated that no reallocation of conservation storage was studied. The feasibility study was cost shared by both states with funding coming from both GI and O&M sources. The model was run on an overnight basis on a CDC machine, but use of a CEAP or PC-486 machine is being evaluated.

REGULATED FLOW PEAK DISCHARGE FREQUENCY ESTIMATES FOR LARGE BASINS

by

Ronald L. Hula[1]

Introduction

Study Purpose. The Southwestern Division (SWD) has been, since the early 1970's, simulating the regulation of the major reservoir systems within the SWD area of responsibility with a computer model (reference 1). The primary purpose of the model is to evaluate alternative plans of regulation from both a hydrologic and an economic perspective. The model is a period of record type program with a routing interval of one day. Residual flood damage computations are an integral part of the model and are based on sequential analysis of the simulated daily hydrographs. These daily hydrographs do not of course define peak discharges with sufficient accuracy at all locations. The recent Arkansas River Reservoir System Studies (reference 4) conducted for the Arkansas River Basin Feasibility Study required the evaluation of alternative system regulation plans. The flood damage calculations were to be performed external to the model by traditional methods which require peak discharge frequency estimates up to the Standard Project Flood (SPF) order of magnitude throughout the basin. Since SWD reservoir system regulation studies must, for practical reasons, be performed using the existing daily model, it was required that a procedure be developed to estimate peak discharges on the basis of simulated daily regulated flows.

Key Issues. The key issues related to the study were that the procedure needed to, 1) be efficient in terms of cost and time, 2) provide estimates up to the SPF order of magnitude, and 3) produce reasonably accurate estimates.

Summary of Findings. It appears, based on evaluation of the study results that, 1) the procedure is efficient and should be incorporated in other SWD models, 2) better verification could be obtained by adjustment of the adopted ratios of peak to average daily flow after an initial system period of record simulation, and 3) the hypothetical flood probability assignments are reasonable in consideration of the size of the basin and the portion of the basin which has experienced peak flows greater than the SPF magnitude during the period of record.

[1]Chief, Hydrologic Engineering Branch, Southwestern Division, U.S. Army Corps of Engineers

Physical Setting and Available Data

Description of Project. The Arkansas River originates in the Rocky Mountains near Leadville, Colorado. It traverses in a general east southeast direction across Colorado, Kansas, Oklahoma and Arkansas to its confluence with the Mississippi River just above Arkansas City, Arkansas. The total contributing drainage area of the Arkansas Basin is 138,000 square miles. The portion of the basin modeled for this study is that area upstream of Little Rock, Arkansas and downstream of the 100th meridian. The excluded portion of the basin upstream of the 100th meridian does not contribute significantly to flood flows into the modeled reservoirs. The modeled area encompasses about 66,000 square miles, 34 storage reservoirs of which 21 are existing projects and 64 stream control points of which 34 are reservoir outflow controls. Figure 1 is a schematic diagram of the model configuration.

Description of Available Data. The data used for this study included the derived period of record flows that are required for input to the regulation simulation model. The period of record is 1940 through 1986, and for each control point in the model, daily flows had been previously developed which represent the total uncontrolled area flow at that point. In addition to the total uncontrolled area flows, there are 82 United States Geological Survey (USGS) stations in the modeled area which have various lengths of record of both daily flows and corresponding peak discharges. These three sources comprise most of the data utilized in the development of the procedure.

Study Approach

Key Assumptions. There are two key assumptions on which the procedure is based. These are that, 1) the ratio of the peak flow to the corresponding average daily flow is a constant for a specific uncontrolled area, and 2) the peak flow at a point under regulated conditions is equal to the sum of the peak flow produced by the uncontrolled area and the average daily flow at that point which is attributable to the releases from the immediate upstream reservoirs. Some degree of error is inherent in these assumptions. First, the maximum 24 hour flow encompassing a peak is most likely not measured by the midnight to midnight average daily flow from the USGS records and secondly, routed reservoir releases are not uniform over a 24 hour period.

Procedure Adopted. The procedures adopted needed to address the development of both period of record and hypothetical flood peaks. The general procedure for developing the period of record peaks was as follows:

1. Optimize the peak to average daily flow ratio for all pertinent USGS stations.

2. Develop a general relationship between uncontrolled drainage area and the optimized peak to average daily flow ratios for various geographic regions.

3. Compute uncontrolled area peaks for the period of record at each of the model control points based on the intervening area at that point and the appropriate peak ratio.

4. Utilize the uncontrolled area peaks as input to the daily regulation simulation model so that flood operations would reflect the additional information. Perform a period of record regulation simulation with only the existing system reservoirs considered operational.

5. Verify by comparing observed peaks with simulated peaks at those control points where peak data is available and for those periods where the upstream control was the same or nearly the same as the current system.

The general procedure for developing hypothetical peaks was as follows:

1. Develop a 3-hour routing interval watershed model which encompasses the entire 66,000 square mile modeled area.

2. Transpose two hypothetical storms critically centered above each reservoir and each control point in the reservoir regulation simulation model. These two storms are based on 40 percent and 50 percent of the Probable Maximum Precipitation (PMP) obtained from Hydrometerological Report 51 No. (HMR 51), reference 3, corresponding to the location of the selected storm center location.

3. Develop the runoff and route and combine hydrographs until the first downstream reservoir is encountered. The result of this step is the development of a total uncontrolled area hydrographs at every control point in the reservoir regulation simulation model for every storm centering and for both storm sizes.

4. Develop average daily flows for all of the transpositions for input to the reservoir regulation simulation model. Also retain the peaks to use as input to the model in a manner similar to that used for the period of record peaks.

5. Operate the reservoir regulation simulation model for all of the hypothetical storm transpositions. At each control point, save the highest peak for each storm size resulting from all of the storm centerings.

6. Assign a probability to each of the storm sizes and plot the two hypothetical storm peak discharges along with the period of record simulated peaks for each control point.

7. Rationalize the assigned hypothetical storm probabilities by the reasonableness of the appearance of the majority of plots and by the reasonableness of the percent of the basin which has experienced peak flow greater than the hypothetical peaks during the period of record.

Computational Methods Used. The general procedures adopted for the study have been outlined above. Explanation of some of those procedures will be given in greater detail below.

The optimization of the ratio of peak to corresponding average daily flow was accomplished for 82 USGS stations within the modeled area of the Arkansas River Basin. The procedure was based on selecting the periods at each station when the flows were either unregulated or essentially unregulated. The optimized ratio was then determined such that the average error in stage between a predicted peak discharge and the corresponding observed peak discharge would be zero. The prediction relationship and the comparison of observations with predictions are shown on Figures 2, 3 and 4 for the Chikaskia River near Blackwell, Oklahoma. Similarly, Figures 5, 6 and 7 are for the Neosho River near Iola, Kansas and Figures 8, 9 and 10 are for the Arkansas River at Van Buren, Arkansas.

The optimized ratios of peak to average daily flow determined for the USGS stations within a geographic region were plotted in correspondence to the effective drainage area at that station as shown on Figure 11. A trend line was sketched through the data points as shown. Additional curves thought to be more representative of individual streams were constructed with the general shape of the trend line but which were closer to the data points developed on the individual stream. An example of this for the Poteau River is also shown on Figure 11. Curves of this type were constructed for all streams in the modeled area. Peak

to average daily flow ratios were then obtained for every control point in the model by use of the appropriate stream curve and the uncontrolled drainage area at that point. These are tabulated in Table 1.

The period of record uncontrolled area flows for each control point were then processed to locate daily hydrograph peaks. Each peak was converted to an instantaneous peak by use of the peak to average daily flow ratio taken from the stream curve for that point. Only those peaks above the partial duration base were retained except that the maximum annual peak was always retained. These peaks, so determined, were used as input to the reservoir regulation simulation model as outlined in the adopted procedure description.

The development of hypothetical storms was accomplished by use of the SWD Watershed Runoff Model. This computer model has an option which allows the storm rainfall and runoff to be analyzed over each cell of a gridded watershed. The runoff from each cell is then lagged, based on input overland flow travel time estimates and the distance to the nearest stream segment. The lagged runoff then becomes inflow to that stream sub-reach. This option allows the use of large watershed sub-areas without the loss of storm pattern definition through the process of determining the average over area storm rainfall. The SWD Watershed Model also has automatic access to the PMP charts in HMR 51. All that is required in the input to define a hypothetical storm is the orientation of the major axis and the latitude and longitude of the storm center.

The SWD Watershed Model was modified so that it would automatically develop hypothetical flood ordinates and the flood peaks for direct input to the reservoir regulation simulation model. In order to establish initial basin conditions for the routing of the hypothetical floods, the regulation simulation model was modified so that general system conditions corresponding to any time of the year could be saved from the period of record simulation. The general conditions so saved were based on an input percent of time exceeded parameter. These conditions were used to establish the initial reservoir storages and the initial stream flows for each of the hypothetical routings.

The SWD Watershed Model was used to develop hypothetical storms at 67 storm center locations within the modeled area. Two storm sizes, 40 percent and 50 percent of PMP, were developed at each storm center.

Study Results

Summary of Results. Period of record peak discharges and hypothetical peaks were generated based on the existing reservoir system and a regulation plan similar to that employed in recent years by execution of the reservoir regulation simulation model in conjunction with the procedures and methods outlined above. After a few trials, it was decided that the most reasonable results were obtained when the hypothetical flood peaks for the two storm sizes were assigned exceedence probabilities of 0.005 and 0.001. The developed peaks, both annual series and partial duration series, were plotted on probability grid for each control point in the model. Three typical results, the Arkansas River at Ralston, Oklahoma, the Neosho River at Iola, Kansas and the Arkansas River at Van Buren, Arkansas are shown on Figures 12, 13 and 14, respectively. It is pointed out again that the assigned exceedence probabilities for the hypothetical peaks were selected to provide a reasonable appearance when all of the control point results were viewed collectively. No attempt was made to sketch a smooth line through the data points. Figure 14 provides clear evidence why an analytic frequency curve is inappropriate when there is a significant degree of upstream regulation. The plot for Iola, Figure 13, shows one extreme period of record event which is the July 1951 flood that centered in eastern Kansas. The hypothetical peaks are of significant help in putting that flood in perspective.

Verification of Results. The verification of results was accomplished in two parts. These were, 1) a comparison of the period of record computed peaks, and 2) an evaluation of the reasonableness of the percent of the modeled area which has experienced peak flows during the period of record which exceeded the hypothetical peaks.

The verification of the period of record computed peaks was performed for as many control points as was possible as follows:

1. A period within the period of record was selected where upstream regulation was the same or nearly the same as the regulation simulation. The regulation simulation again was based on the existing system.

2. The period selected was further shortened to only include that time where the plan of regulation above that point was similar to the plan employed in the simulation.

3. The selected period was further shortened to only include that time when USGS peak data was available.

4. The maximum annual average daily flows, both computed and observed, were plotted on probability grid for the selected period. The purpose of this step is

to provide a basis for judging how well the flow data, the regulation plan and the regulation simulation model approximate the flow observations during the selected period.

5. The peak annual series discharges, both computed and observed, were plotted on probability grid for comparison.

6. The peak partial duration series discharges, both computed and observed, were plotted on probability grid for comparison. This step is not always possible as some USGS stations do not have partial duration series records.

These results of these steps are shown for six control points on Figures 15 through 28. In general, the verification is good for those control points with larger effective drainage area and correspondingly lower ratios of peak to average daily discharge (refer to Table 1 for the effective drainage area and the adopted ratio for a particular control point). The verification plots for the control point on the Neosho River at Americus, Kansas are shown on Figures 21 and 22. This is the poorest verification of all of the control points. The adopted ratio for this control point is 2.5 based on the uncontrolled drainage area of 94 square miles. Examination of Figure 22 indicates that the verification probably would have been fairly good if the adopted ratio had been about 1.3.

The verification of the reasonableness of the probability assignments for the hypothetical events was accomplished by the following analysis.

1. It was estimated that approximately 5,000 square miles of the 66,000 square mile modeled area has experienced one or more occurrences of peak discharge which exceeded the hypothetical peaks during the period of record. This probably would also be true for an even longer period, but general stream gage coverage did not begin until about 1940. The period of record is 47 years, however it is not considered unreasonable to assume that the 5,000 square mile area would apply to a period of 60 years or more. The risk that any location within the modeled area would experience one or more events during a period in the range of 60 years is estimated as 5,000/66,000 or about 0.08.

2. The binomial expression for risk is given by

$$R = 1-(1-P)^N \quad \text{(equation 10-3, reference 2)}$$

where: R = risk of one or more exceedences of an event
P = probability of the event
N = the number of trials

Solution of this equation for P under the assumption that R=0.08 is as follows for several values of N.

R	N	P
.08	50	.0017
.08	60	.0014
.08	70	.0012
.08	80	.0010

Conclusion

Discussion of Conclusions. The procedures appear to have merit for increasing the accuracy of flood damage computations performed by the SWD Reservoir Regulation Simulation Model. In the past, these computations have been based on the peaks of the average daily flow hydrographs produced by the model. While this is of sufficient accuracy for the larger uncontrolled drainage areas within the models, the smaller areas would benefit from the approach. It appears also that fairly reliable annual and partial duration series peak discharge data points can be determined up to the order of magnitude of the SPF, at least for the larger uncontrolled areas. The results of the study indicate there was considerable error for some of the smaller uncontrolled areas.

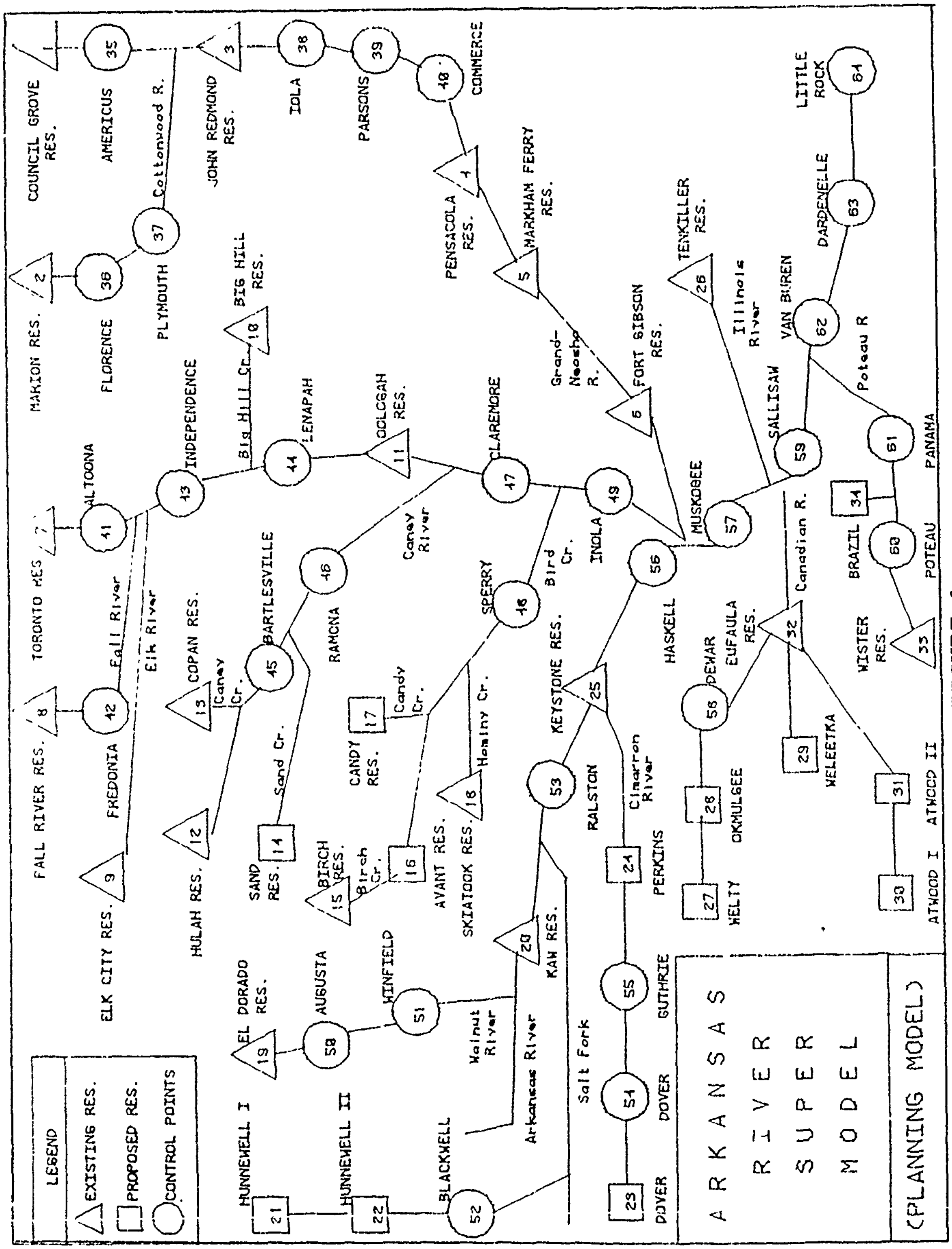
Hindsight Observations. In retrospect, it appears that the procedure should have been expanded to include steps to adjust the adopted ratios of peak to average daily flows after an initial verification step was performed. It is believed that much better subsequent verification could have been obtained for the smaller areas.

It appears that the value obtained from optimizing the ratios of peak to average daily flow to obtain zero average error in the stage predictions is not warranted. This requires additional effort in collecting and processing stage discharge curves for each station to be analyzed. It would probably have been just as satisfactory if the ratios had been optimized to produce zero average error in the predicted discharge. This additional effort would be even less important if the adopted ratios were to be adjusted after an initial verification of peak discharges.

TABLE 1

ADOPTED PEAK TO AVERAGE DAILY FLOW RATIOS

STREAM	CONTROL POINT	EFFECTIVE DRAINAGE AREA	RATIO
Arkansas River	Ralston	4,735	1.05
Arkansas River	Haskell	967	1.32
Arkansas River	Muskogee	3,136	1.07
Arkansas River	Sallisaw	5,087	1.04
Arkansas River	Van Buren	6,685	1.04
Arkansas River	Dardanelle	9,808	1.04
Arkansas River	Little Rock	13,000	1.04
Bird Creek	Sperry	78	1.59
Caney River	Bartlesville	228	1.40
Caney River	Ramona	581	1.19
Chikaskia River	Blackwell	358	1.97
Cimarron River	Dover	630	1.85
Cimarron River	Guthrie	1,500	1.37
Cimarron River	Perkins	1,900	1.27
Cottonwood R.	Florence	554	1.50
Cottonwood R.	Plymouth	1,540	1.20
Deep Fork River	Dewar	250	1.34
Fall River	Fredonia	242	1.89
Neosho River	Americus	94	2.50
Neosho River	Iola	803	1.36
Neosho River	Parsons	1,890	1.15
Neosho River	Commerce	2,861	1.07
Poteau River	Poteau	247	1.31
Poteau River	Panama	973	1.10
Verdigris River	Altoona	408	1.64
Verdigris River	Independence	943	1.31
Verdigris River	Lenepah	1,690	1.17
Verdigris River	Claremore	821	1.36
Verdigris River	Inola	1,371	1.22
Walnut River	Augusta	205	1.98
Walnut River	Winfield	1,633	1.19



LEGEND
EXISTING RES.
PROPOSED RES.
CONTROL POINTS
COUNCIL GROVE RES.
AMERICUS
Cottonwood R.
JOHN REDMOND RES.
IOLA
PARSONS
COMMERCE
PENSACOLA RES.
MARKHAM FERRY RES.
MARION RES.
FLORENCE
PLYMOUTH
BIG HILL RES.
Big Hill Cr.
TENKILLER RES.
Illinois River
DARDENELLE
LITTLE ROCK
VAN BUREN
Poteau R
Grand-Neosho R.
FORT GIBSON RES.
SALLISAW
PANAMA
TORONTO RES.
ALTOONA
INDEPENDENCE
LENAPAH
OOLOGAH RES.
CLAREMORE
MUSKOGEE
INOLA
Bird Cr.
Caney River
Fall River
Elk River
BARTLESVILLE
RAMONA
COPAN RES.
Caney Cr.
SPERRY
HASKELL
Canadian R.
BRAZIL
POTEAU
FALL RIVER RES.
FREDONIA
KEYSTONE RES.
DEWAR
EUFAULA RES.
WISTER RES.
Sand Cr.
CANDY RES.
Candy Cr.
Hominy Cr.
Cimarron River
WELEETKA
ATWOOD II
ELK CITY RES.
HULAH RES.
SAND RES.
BIRCH RES.
Birch Cr.
AVANT RES.
SKIATOOK RES.
RALSTON
PERKINS
OKMULGEE
KELTY
ATWOOD I
EL DORADO RES.
AUGUSTA
WINFIELD
KAW RES.
Walnut River
Arkansas River
Salt Fork
GUTHRIE
DOVER
HUNNEWELL I
HUNNEWELL II
BLACKWELL
ARKANSAS RIVER SUPER MODEL
(PLANNING MODEL)

FIGURE 1

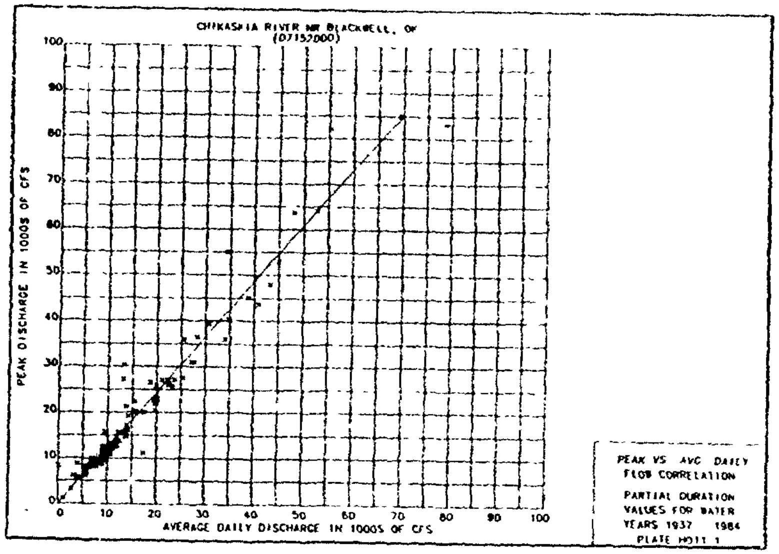


FIGURE 2

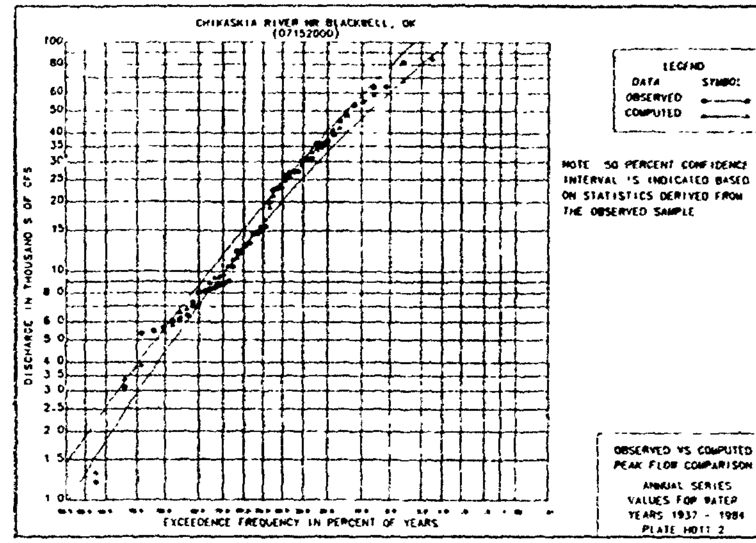


FIGURE 3

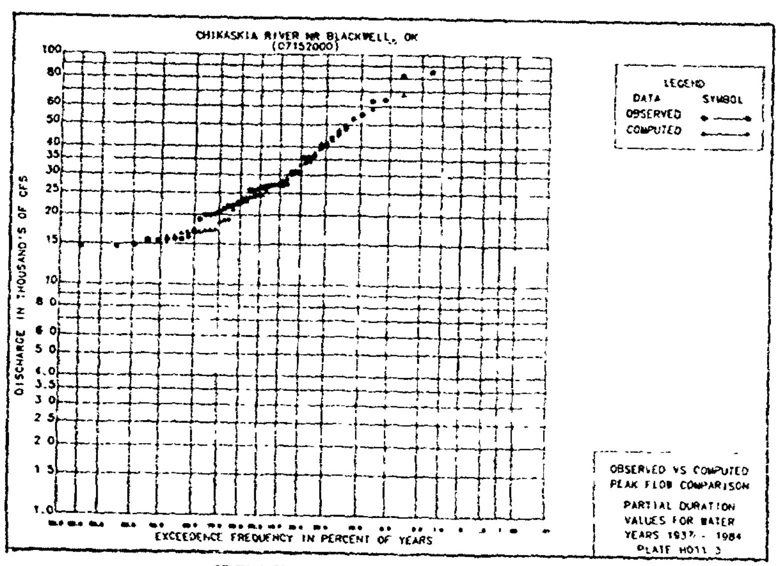


FIGURE 4

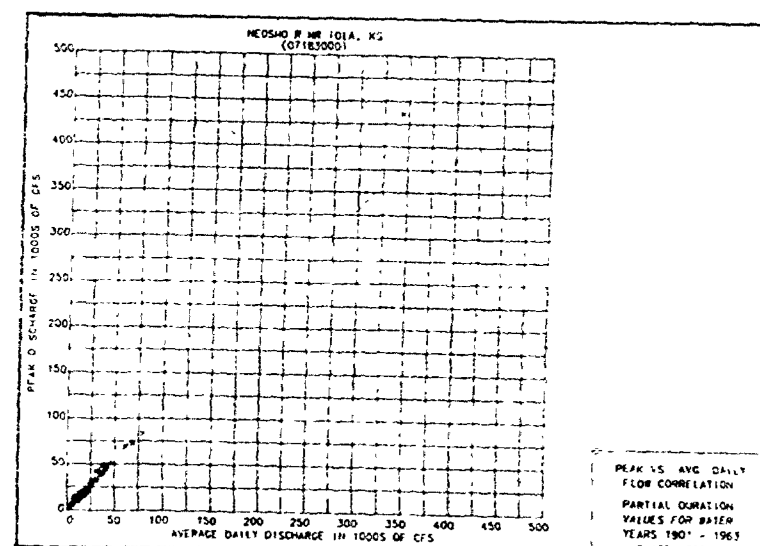


FIGURE 5

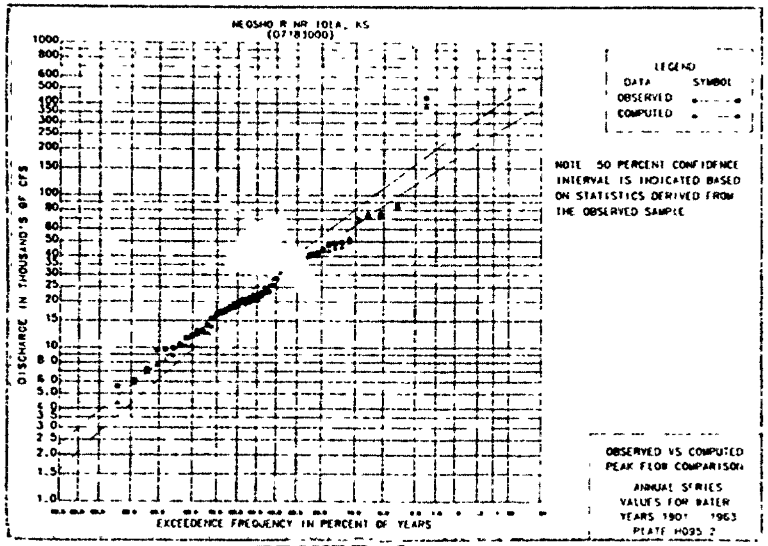


FIGURE 6

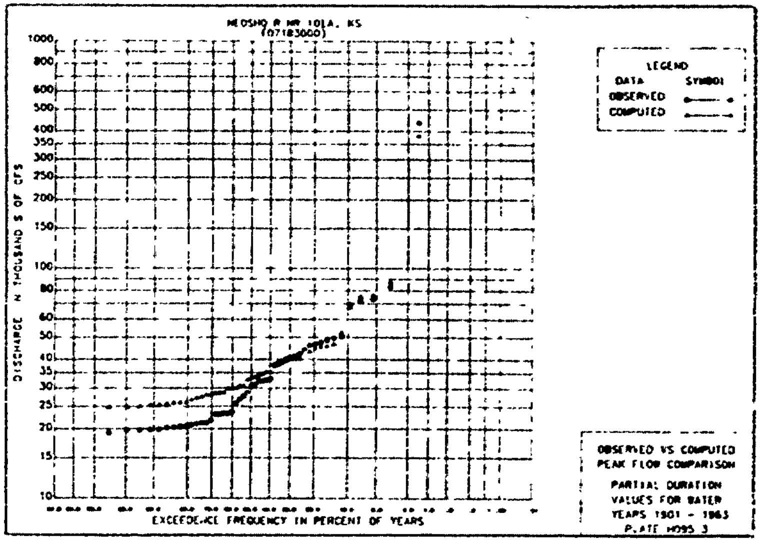


FIGURE 7

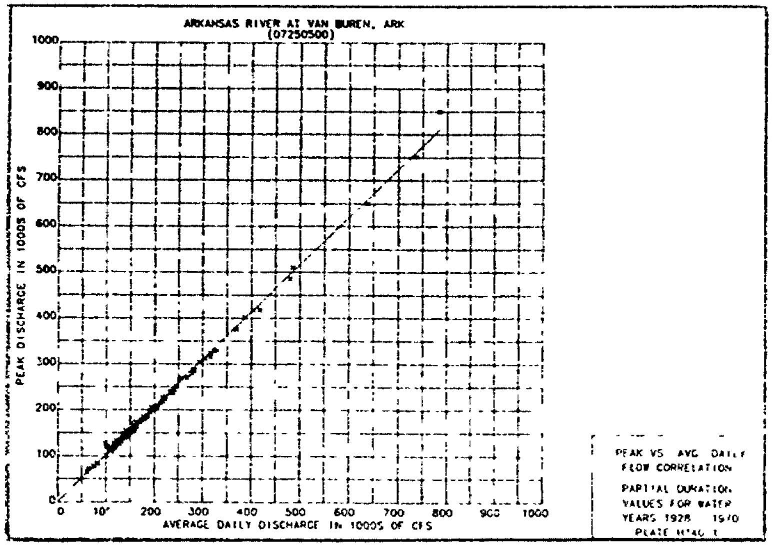


FIGURE 8

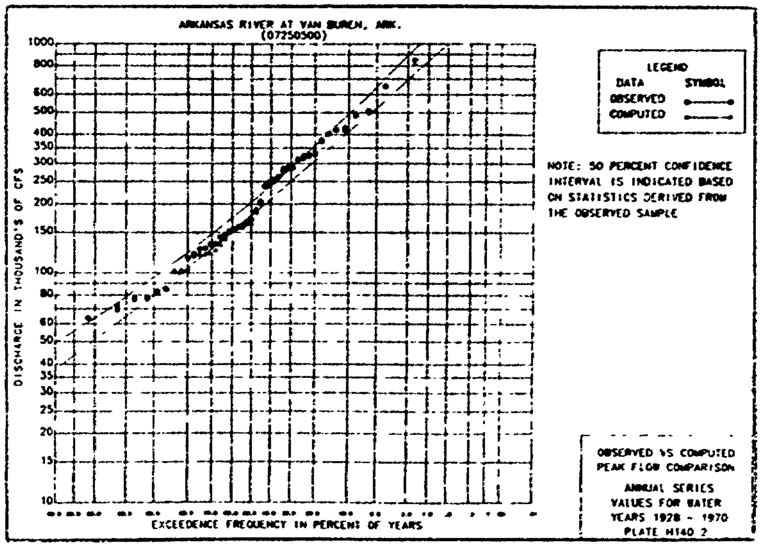


FIGURE 9

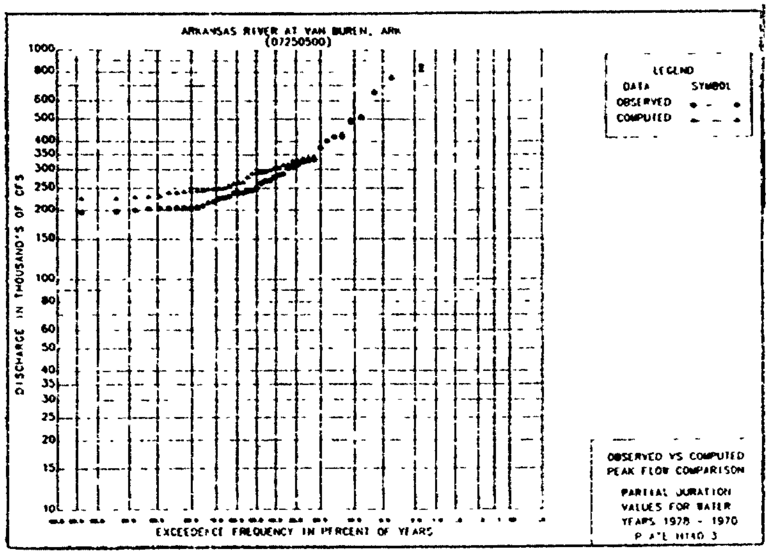


FIGURE 10

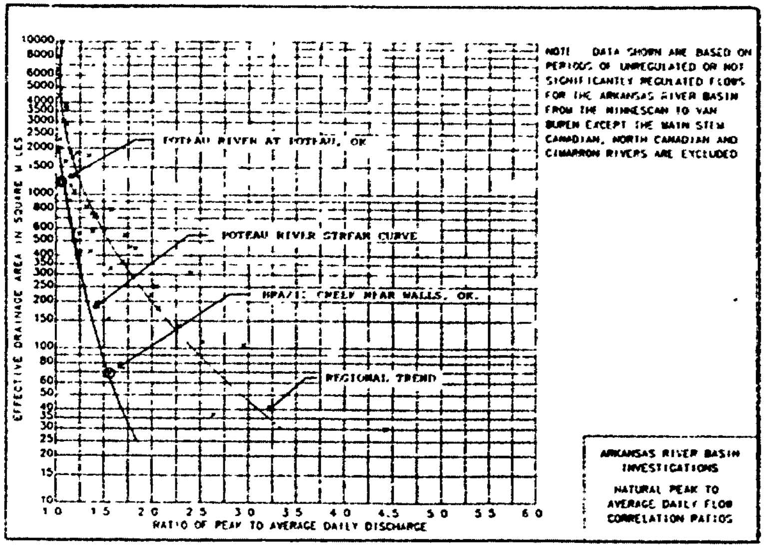


FIGURE 11

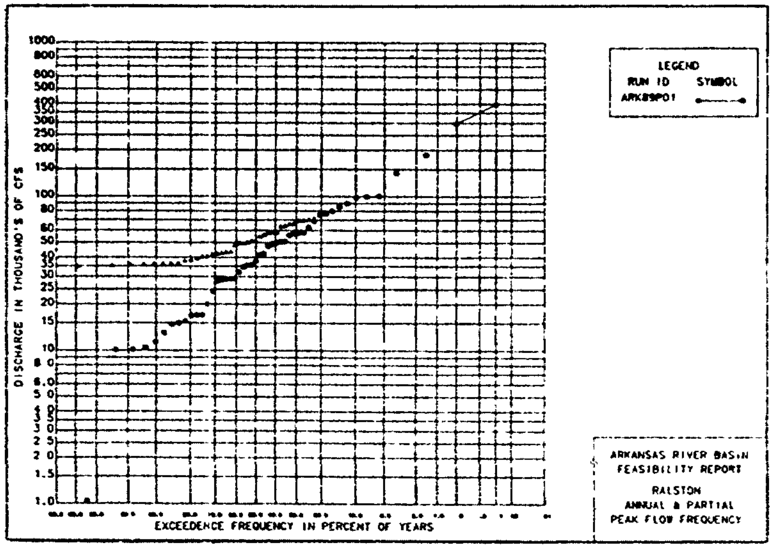


FIGURE 12

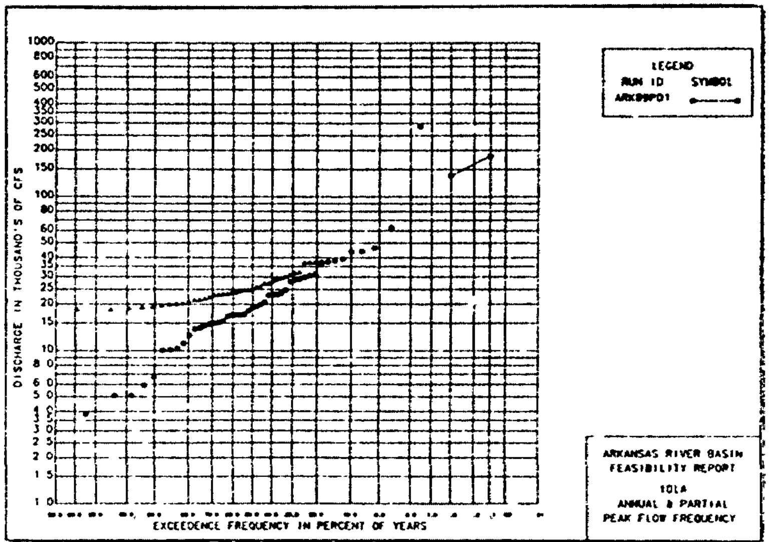


FIGURE 13

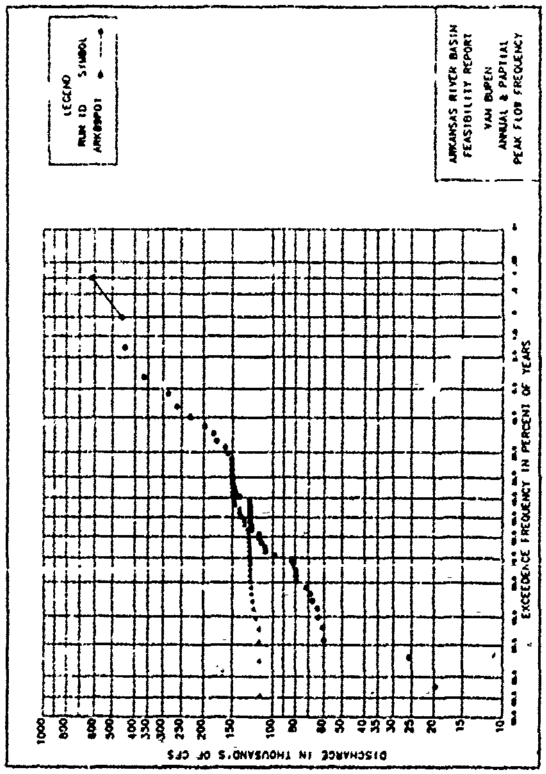


FIGURE 14

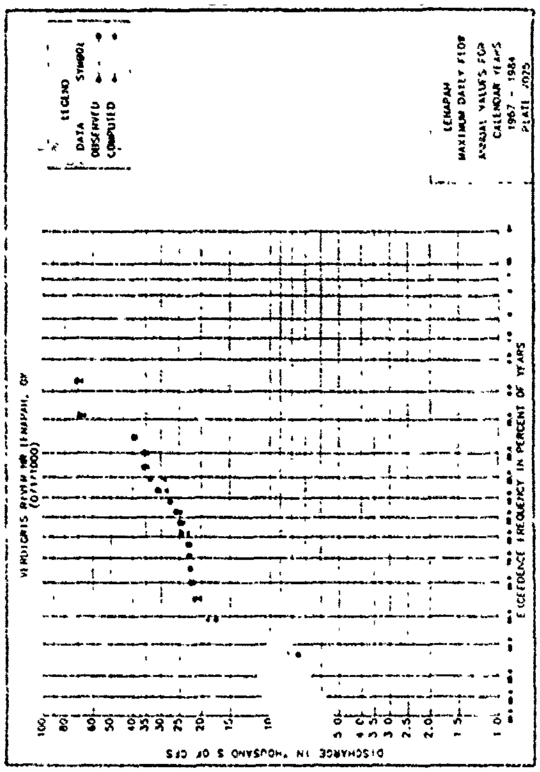


FIGURE 15

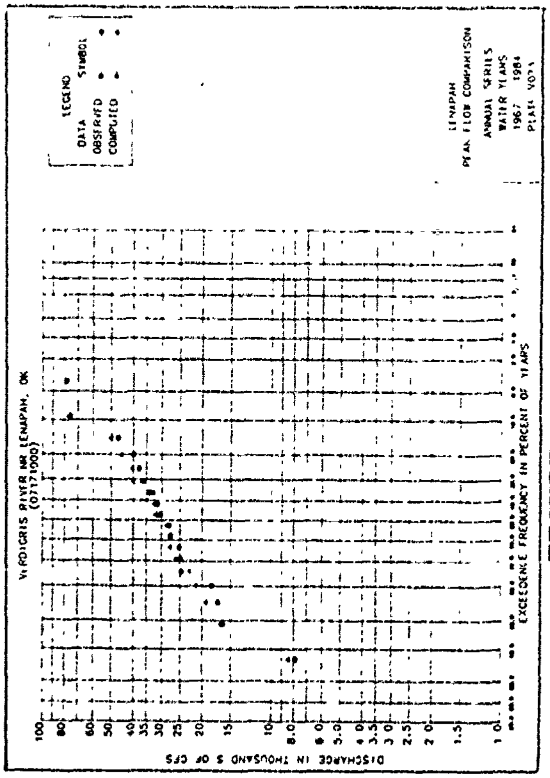


FIGURE 16

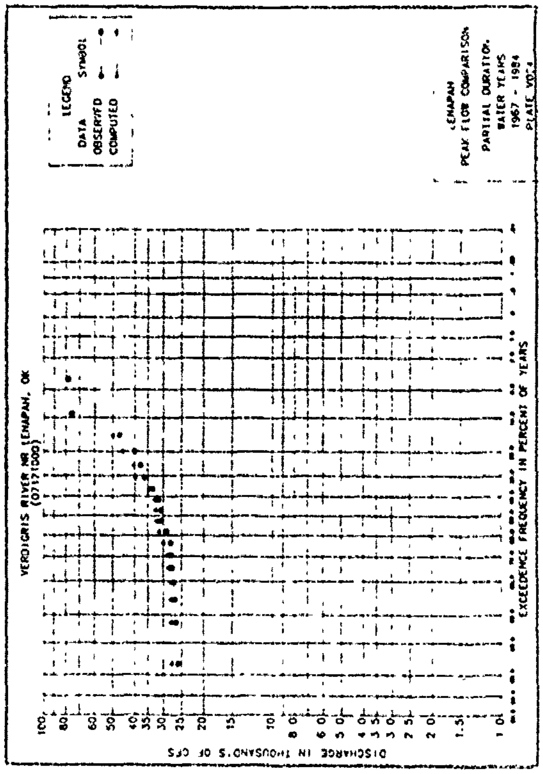


FIGURE 17

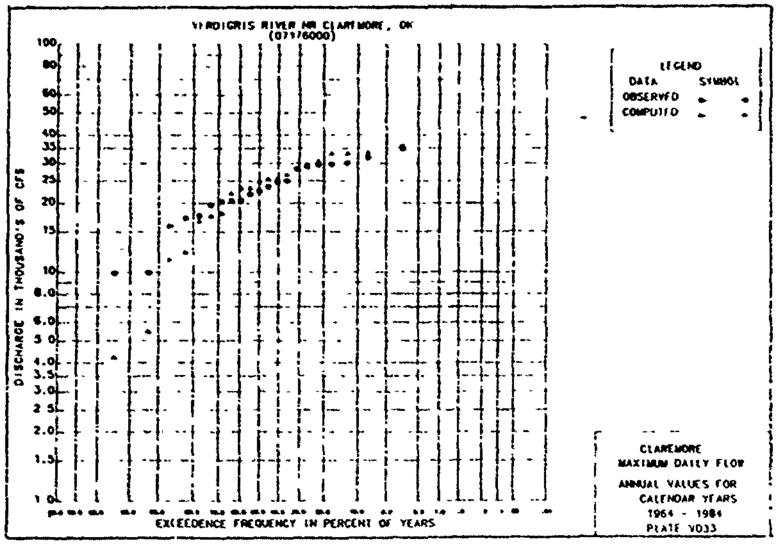


FIGURE 18

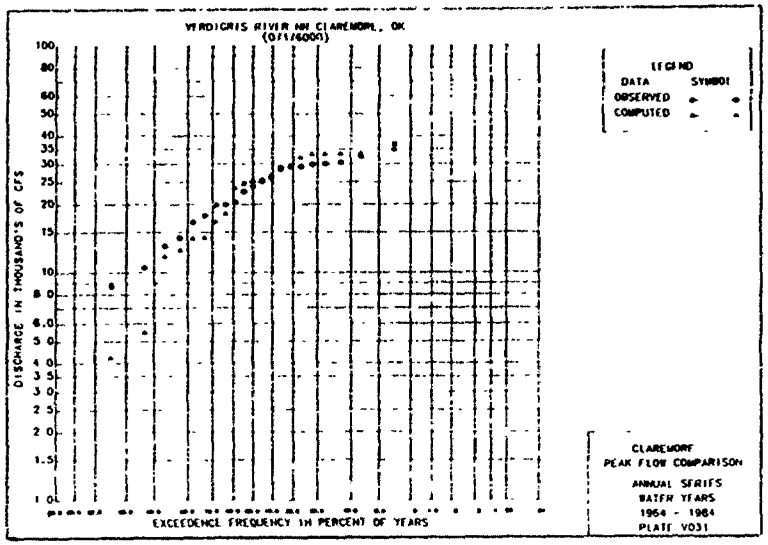


FIGURE 19

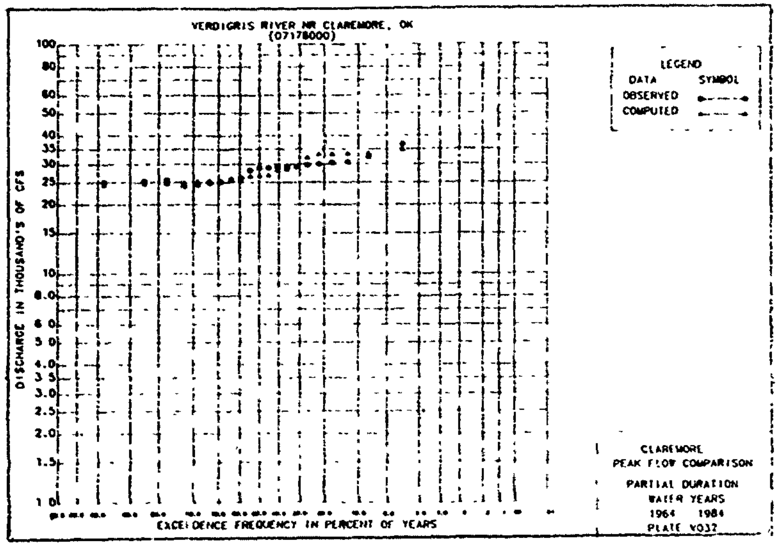


FIGURE 20

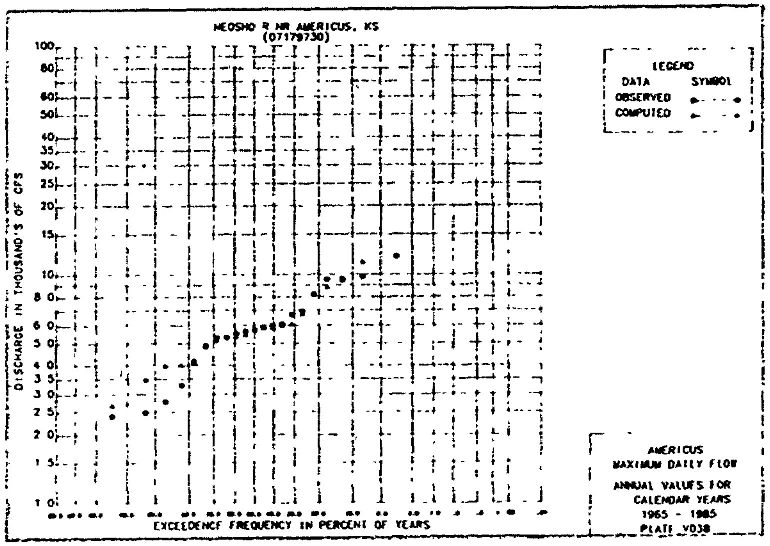


FIGURE 21

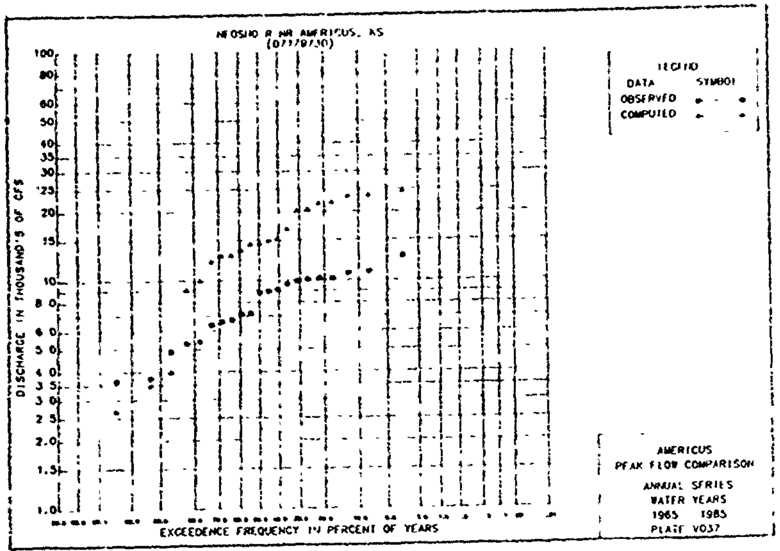


FIGURE 22

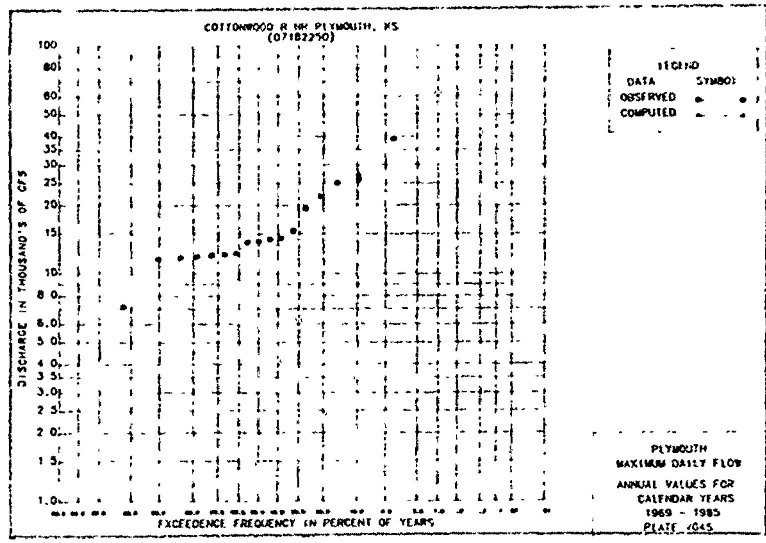


FIGURE 23

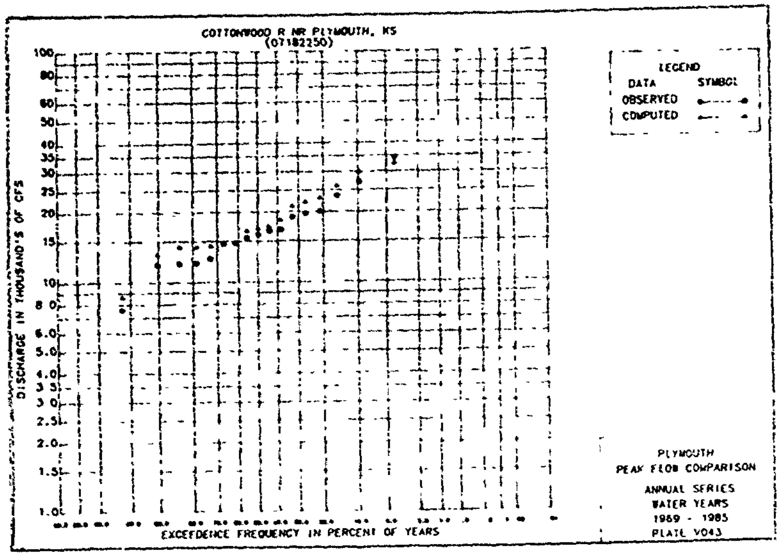


FIGURE 24

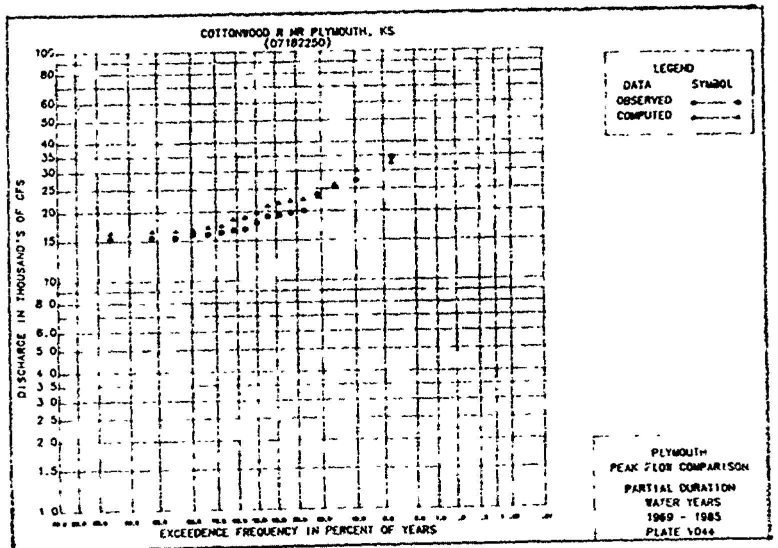


FIGURE 25

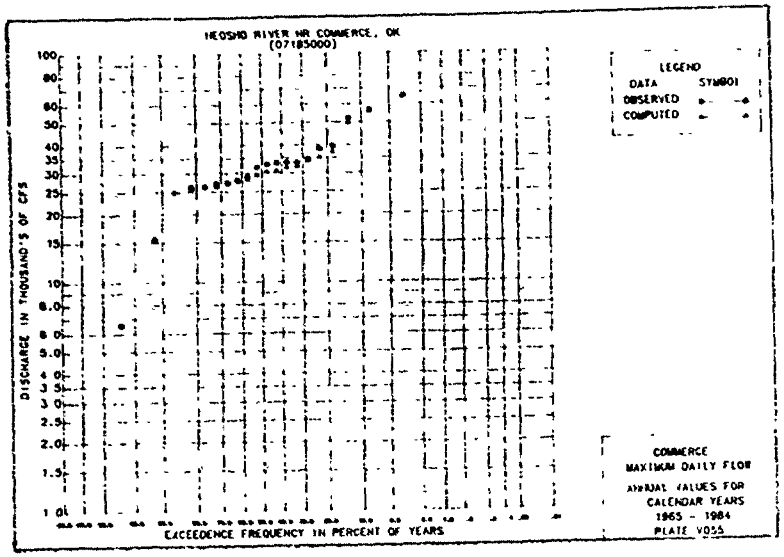


FIGURE 26

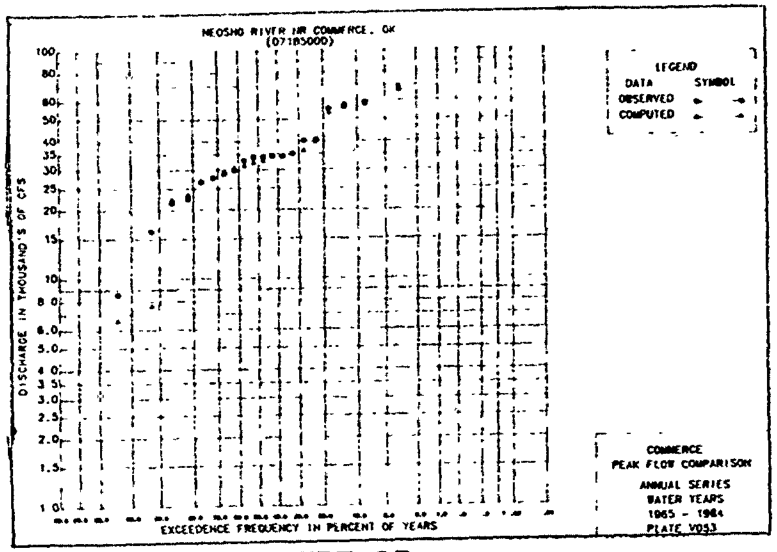


FIGURE 27

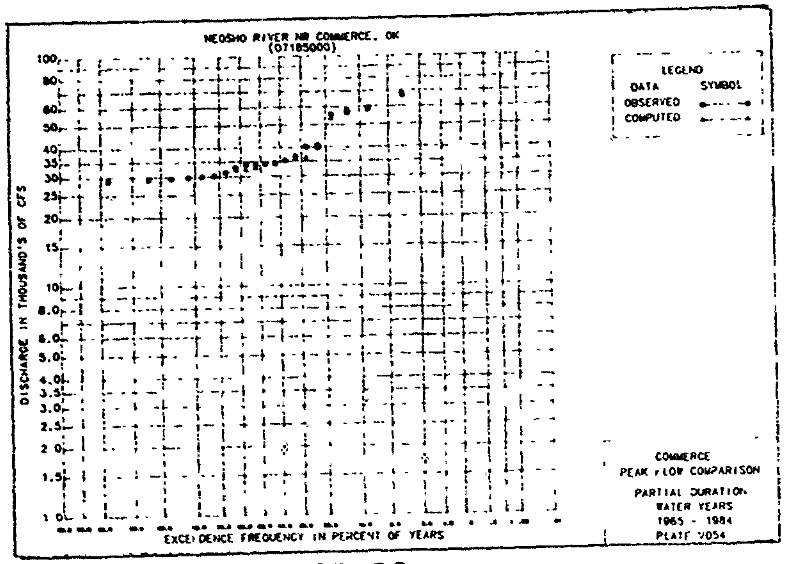


FIGURE 28

References

1. Hula, Ronald L., Southwestern Division Reservoir Regulation Simulation Model, ASCE National Workshop on Reservoir Systems Operations, Boulder, Colorado (1979)

2. Interagency Advisory Committee on Water Data, Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Subcommittee (rev 1981)

3. Schreiner, Louis C. and Riedel, John T., Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 51, National Weather Service, National Oceanic and Atmospheric Administration, U. S. Department of Commerce (1978)

4. Word, Clinton E., Arkansas River - Reservoir System Studies, Corps of Engineers Hydrologic Engineering Center Workshop on Hydrologic Studies in Support of Project Functions, Angel Fire, New Mexico (1990)

Regulated Flow Peak Discharge Frequency Estimates For Large Basins.

by

Ronald L. Hula

SUMMARY OF DISCUSSION BY BRUCE C. BEACH

In response to a question, the author stated that the result of the feasibility report was negative, but that the model would be used over and over again in response to pressure from various interest groups. He added that another use of the model would be in response to criticism from floods. The public isn't aware of all the floods that didn't happen; that were prevented by the system. Use of the model would help demonstrate to the public system benefits.

REEVALUATION OF FREQUENCY OF REGULATED FLOWS ON THE AMERICAN RIVER AT SACRAMENTO

by

Russell P. Yaworsky[1]

Introduction

Purpose. Sacramento is a rapidly growing metropolitan area located at the junction of the Sacramento and American Rivers, two California rivers with a high flood potential. The American River has experienced several large flood events within the past 35 years which strained the operation of the existing flood control system. As a result, an effort was made to update the hydrology of the American River and to evaluate both the existing flood control system and measures to upgrade the system. This paper briefly discusses the study approach and analysis.

History. Folsom Dam, approximately 25 miles upstream of the City of Sacramento, and its associated downstream levees are the sole flood control features in the basin (see Figure 1). The reservoir space in Folsom Lake dedicated to flood control is based on the Reservoir Design Flood (RDF), which was computed as the flood resulting from the largest rainstorm of record within the region (December 1937). Using the RDF as a guide, the dam was built in 1955 to provide a maximum of 400,000 acre-feet of flood control space with an objective outflow of 115,000 cfs. The downstream levees are currently considered capable of safely accommodating sustained flows of 115,000 cfs.

In February 1986, major storms in northern California caused record flood flows in the American River Basin. A peak outflow of 130,000 cfs from Folsom Dam exceeded the objective release of 115,000 cfs for a period of 48 hours. Prior to 1986, it was believed that Folsom could provide up to a 120-year level of protection and that a flow of 115,000 cfs would not be exceeded more than once in 100 years, on the average. However, in addition to 1986, Folsom peak releases equalled 110,000 cfs in February 1963, 115,000 cfs in December 1964, and would have equalled 115,000 cfs in December 1955 except that Folsom storage was well below the bottom of the flood pool because filling began that year.

Summary of Findings. The purpose of the study was to review and update the hydrology of the American River. This was accomplished by developing current condition unregulated and regulated discharge-frequency relationships. The updated hydrology showed that Folsom Dam is capable of controlling to the 63-year event with surcharging and without having to release more than 115,000 cfs. This reduction in the level of protection provided is due primarily to the additional 30 years of record. Floods of design magnitude are now estimated to occur much more frequently. Since completion of Folsom Dam, three floods have exceeded the volume of the RDF (December 1955 and 1964, and February 1986). Seven of the ten largest recorded events have occurred since 1950 (see Figure 2).

Identified measures to help increase the level of downstream flood protection included (1) increasing the flood control storage in Folsom, (2) increasing the downstream levee and channel flood carrying capacity, (3) using existing upstream reservoir space for flood control, (4) modifying Folsom Dam to permit increased releases, and (5) constructing new upstream flood control storage.

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[1] Hydraulic Engineer, Sacramento District, U.S. Army Corps of Engineers

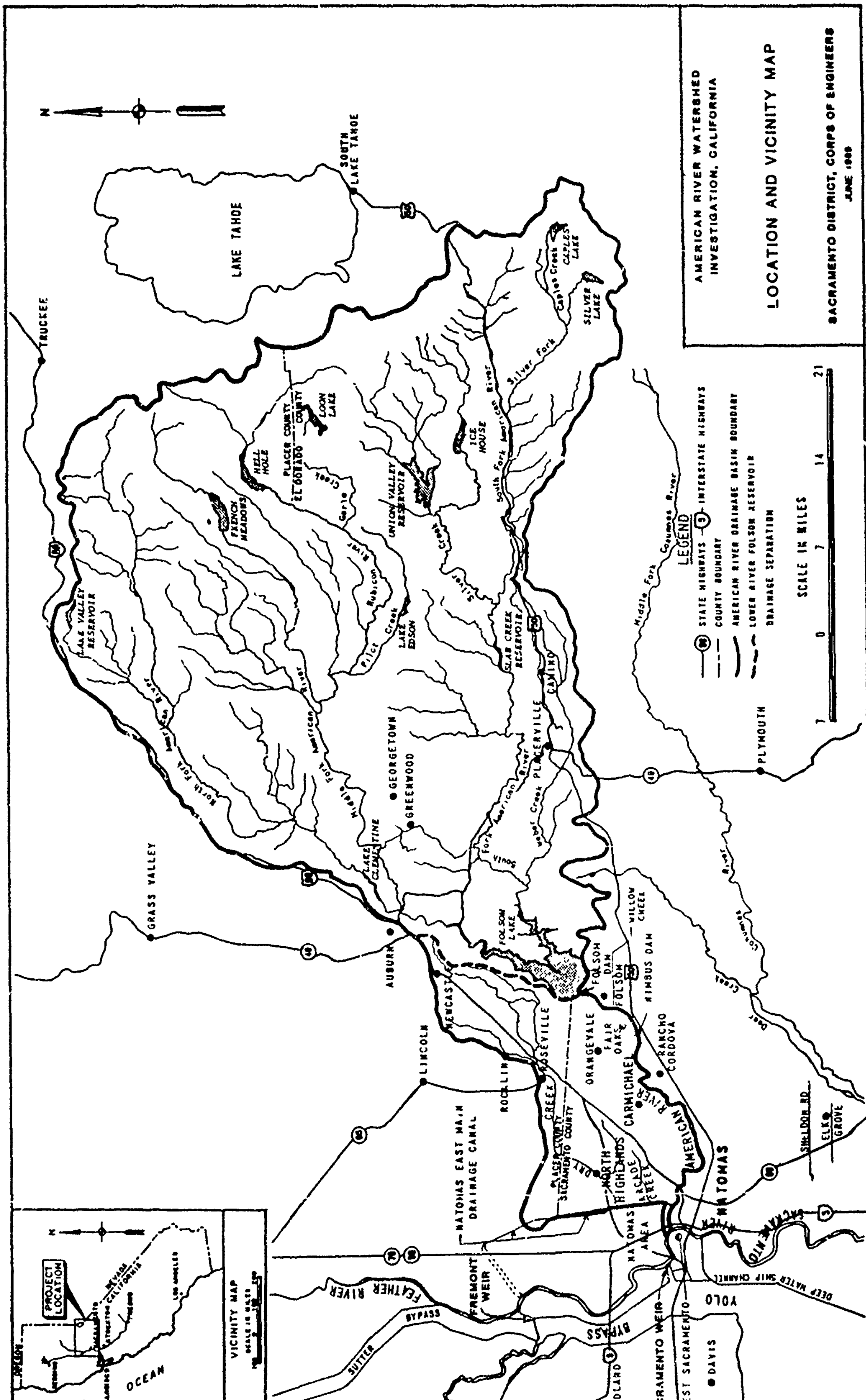

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA
LOCATION AND VICINITY MAP
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JUNE 1969
LEGEND
STATE HIGHWAYS
INTERSTATE HIGHWAYS
COUNTY BOUNDARY
AMERICAN RIVER DRAINAGE BASIN BOUNDARY
LOWER RIVER FOLSOM RESERVOIR DRAINAGE SEPARATION
SCALE IN MILES
7 0 7 14 21
VICINITY MAP
PROJECT LOCATION
PACIFIC OCEAN
NEVADA
CALIFORNIA
LAKE TAHOE
SOUTH LAKE TAHOE
TRUCKEE
GRASS VALLEY
AUBURN
LINCOLN
ROCKLIN
ROSEVILLE
NEWCASTLE
FOLSOM
FOLSOM LAKE
FOLSOM DAM
NIMBUS DAM
ORANGEVALE
FAIR OAKS
CARMICHAEL
RANCHO CORDOVA
NORTH HIGHLANDS
PLACERVILLE
CAMINO
PLYMOUTH
GEORGETOWN
GREENWOOD
LAKE CLEMENTINE
LAKE VALLEY RESERVOIR
FRENCH MEADOWS
HELL HOLE
LOON LAKE
UNION VALLEY RESERVOIR
ICE HOUSE
SLAB CREEK RESERVOIR
LAKE EDSON
SILVER LAKE
CAPLES LAKE
PLACER COUNTY
EL DORADO COUNTY
SACRAMENTO COUNTY
North Fork American River
Middle Fork American River
South Fork American River
Rubicon River
Silver Fork
Silver Creek
Pilot Creek
Gerle Creek
Caples Creek
Weber Creek
Middle Fork Cosumnes River
Cosumnes River
Deer Creek
WILLOW CREEK
DRY CREEK
ARCADE CREEK
AMERICAN RIVER
NATOMAS EAST MAIN DRAINAGE CANAL
NATOMAS AREA
NATOMAS
SACRAMENTO RIVER
FEATHER RIVER
FREMONT WEIR
SUTTER BYPASS
YOLO BYPASS
SACRAMENTO WEIR
DEEP WATER SHIP CHANNEL
WEST SACRAMENTO
WOODLAND
DAVIS
ELK GROVE
SHELDON RD

Figure 1

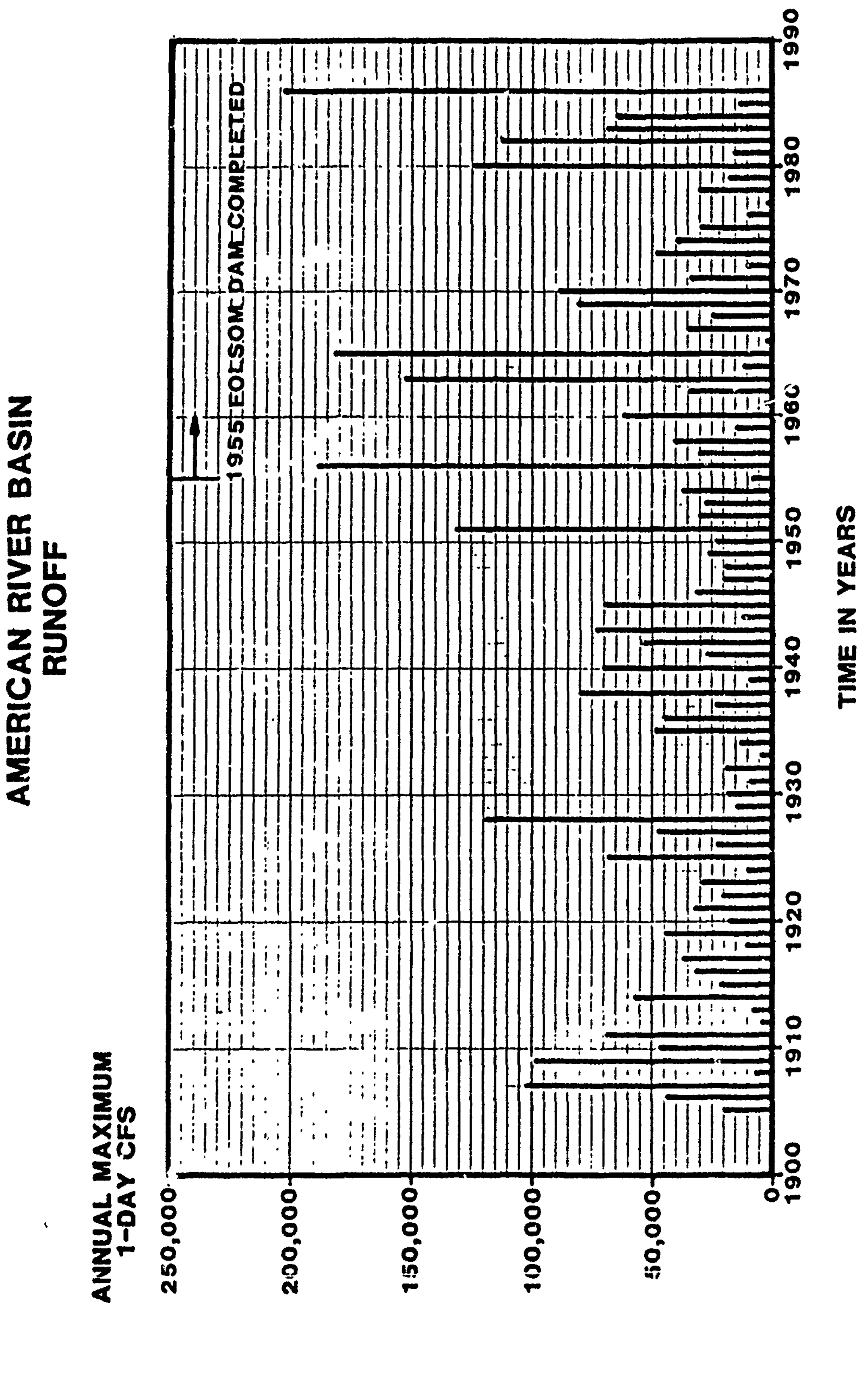

AMERICAN RIVER BASIN
RUNOFF
ANNUAL MAXIMUM
1-DAY CFS
250,000
200,000
150,000
100,000
50,000
0
1900
1910
1920
1930
1940
1950
1960
1970
1980
1990
1955 FOLSOM DAM COMPLETED
TIME IN YEARS

Figure 2

Study Area

Basin Topography. The American River Basin encompasses about 2,100 square miles. The headwaters of the basin originate in the Sierra Nevada Mountains at an elevation of 10,400 feet and flow generally westward to the Sacramento River (see Figure 1). The basin is drained by three large branches, the North, Middle and South Forks. The three forks unite into one main channel within the reservoir area. The elevation of the basin where the American River flows into the Sacramento River is near sea level. The average basin slope is 80 feet per mile. The upper third of the basin has been intensely glaciated and is alpine in character with bare granite peaks and ridges. The middle third is intensely dissected by profound deep canyons, while the lower third consists of low rolling mountains and foothills. Major development is limited to the lower third of the basin.

Storms and Floods. The American River Basin lies on the seaward face of the Sierra Nevada which rise directly across the path of storms moving inland from the Mid-Pacific Ocean. The low barrier of the Coast Range, which intervenes between the ocean and the Sierra Nevada, is pierced by the large San Francisco Bay Gap westward from the basin so that considerable volumes of moist maritime air reach the basin at low levels. The major storm events are characteristically rain and snow and are generally composed of a series of storms which dump a large volume of water into the system. Precipitation normally falls as snow above the 5,000 foot level, but during extremely warm winter storms rain has fallen over the entire basin melting some of the snow, and at times stripping most of the snow from the basin.

The annual precipitation is concentrated almost entirely during the winter storm season from November through March. Figure 3 is a histogram of mean monthly precipitation at selected stations. Summer thunderstorms which occur over small areas barely affect the mainstem flows. In addition, spring snowmelt floods are characterized by low peaks, long flow durations and large volumes of runoff, and normally do not present a flood problem because of the relatively large release capability at Folsom.

The high flood potential in the basin is attributable to the storm track, orographic effects and geology of the Sierra Nevada Mountains, and by the concurrence of flows from the three main branches in the reservoir area.

Study Approach

An analysis of available flow data was needed to determine the flood potential of the basin. The last statistical analysis of the American River was done in 1961 and included flow data for water years 1905-1956. An additional 30 years of record, up to and including 1986, was included in the present analysis. An attempt was made to estimate historic peaks outside of the gaged period, prior to the 1900's. However, extensive hydraulic mining for gold on the lower American River, and in adjacent basins, had since significantly altered the flow regime in the Sacramento Valley making reasonable estimates of the magnitude of these events difficult.

Unregulated Frequency Analysis. Development and analysis of unregulated flows were needed to provide a basis for evaluation of the existing system and any alternatives considered. Unregulated mean daily flow was determined by computing daily reservoir holdouts (change in storage in cfs) and combining them with the recorded regulated flow at the Fair Oaks gage just downstream of Folsom Dam. The reservoir holdouts account for the effects of Folsom and the largest upstream reservoirs including French Meadows, Hell Hole, Loon Lake, Union Valley and Ice House (see Figure 1). The computed flows updated the previous period of recorded natural flow, water years 1905-56, to the long-term records of 1905-1986. This new streamflow record

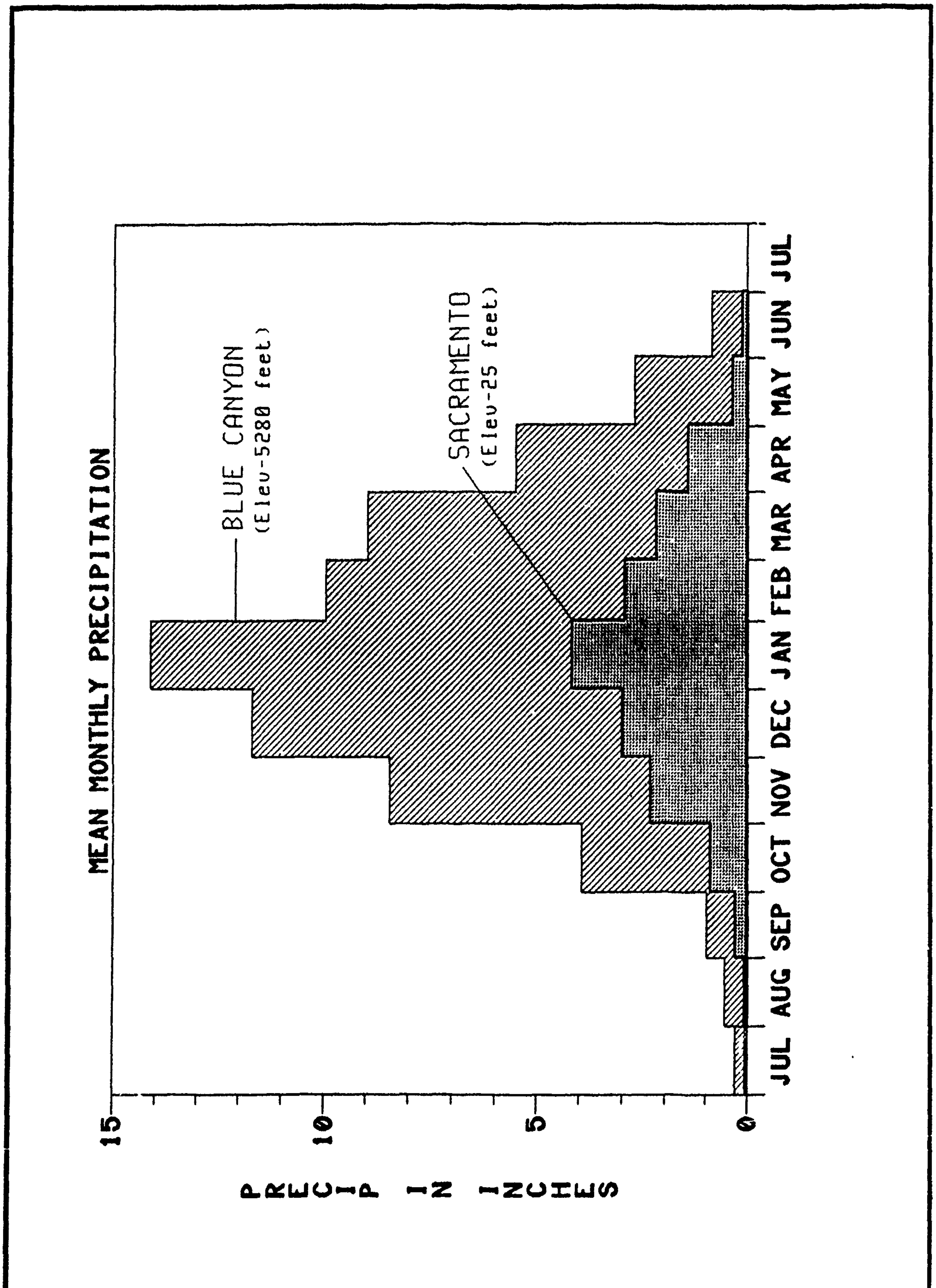

MEAN MONTHLY PRECIPITATION
BLUE CANYON
(Elev-5280 feet)
SACRAMENTO
(Elev-25 feet)
PRECIP IN INCHES
15
10
5
0
JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL

Figure 3

was used to develop annual maximum volume-frequency relationships for durations of 1-, 3-, 5-, 7-, 10-, 15-, and 30-days at Fair Oaks. Computed statistics for the analytical frequency curves were adjusted to assure a smooth, consistent family of curves. The unregulated rain flood frequency curves are shown on Figure 4.

Flow-Frequency - Project Conditions. Evaluation of the existing flood control system required a flow-frequency analysis for the present project conditions at Fair Oaks. Estimated affects of storage in the upstream reservoirs and of Folsom operation were included in the derivation of the frequency curve for existing conditions (see Figure 5). The 31 years of actual recorded flow data, since construction of the dam, were used to define the plotting positions of flows more frequent than the 50-year exceedence interval. For less frequent flows, or to extrapolate beyond the historical record, hypothetical flood hydrographs were developed and routed through Folsom. The unregulated flow volumes, see Figure 4, were used to generate the hypothetical inflow hydrographs for each exceedence interval.

The shape of the hypothetical inflow hydrographs was derived from a balanced 200-year flood series, see Figure 6, that was patterned after the Probable Maximum Flood (PMF) developed in 1980 to evaluate the adequacy of the Folsom spillway.

The 30-day series of flood waves, see Figure 6, typifies the major storm events in the region. Many floods are preceded and/or followed by other storms. Operational studies must therefore not only consider the largest flood event in the series, but also the potential for smaller floods infringing on the remaining flood space.

Hypothetical reservoir routings of various wave sequences were done to find the most critical scenario. The sequence of flood waves can vary, as long as the volume relationships are preserved. Beginning the routings with the large wave first was determined to be the most critical. The routings indicated that preceding the large flood wave with a smaller wave, in effect, improved operation by allowing Folsom to pass the small wave and a significant portion of an initial flood space encroachment contingency. The encroachment of 80,000 acre-feet into the flood space was applied to account for uncertainties in realtime operation that have been experienced during 30-years of actual operation. This uncertainty is due to the basin's potential for generating a large volume of inflow in a relatively short time and because Folsom Dam cannot pass these high inflows soon enough.

A review of historical floods also showed that about 50,000 acre-feet of effective upstream storage space would be available during major floods up through the 100-year frequency. This volume was gradually shaved from the rising limb of the hypothetical inflow hydrograph to simulate impounding by the upstream reservoirs. No reduction in inflow was made for floods larger than the 100-year event, because it was assumed that preceding storms would have been sufficient to fill the upstream storage space, or that the space available would have been ineffective. This was deemed a reasonable assumption since both situations have occurred at times in the past.

Flood Control Measures

The previous sections have shown that the existing flood control system on the American River provides a lower level of protection than designed. Several flood control measures were considered to provide additional flood protection. These measures are discussed in the following paragraphs.

Existing Upstream Storage. There is a total of about 820,000 acre-feet of storage capacity in the upstream reservoirs; however, all of these reservoirs are designed for water supply and/or

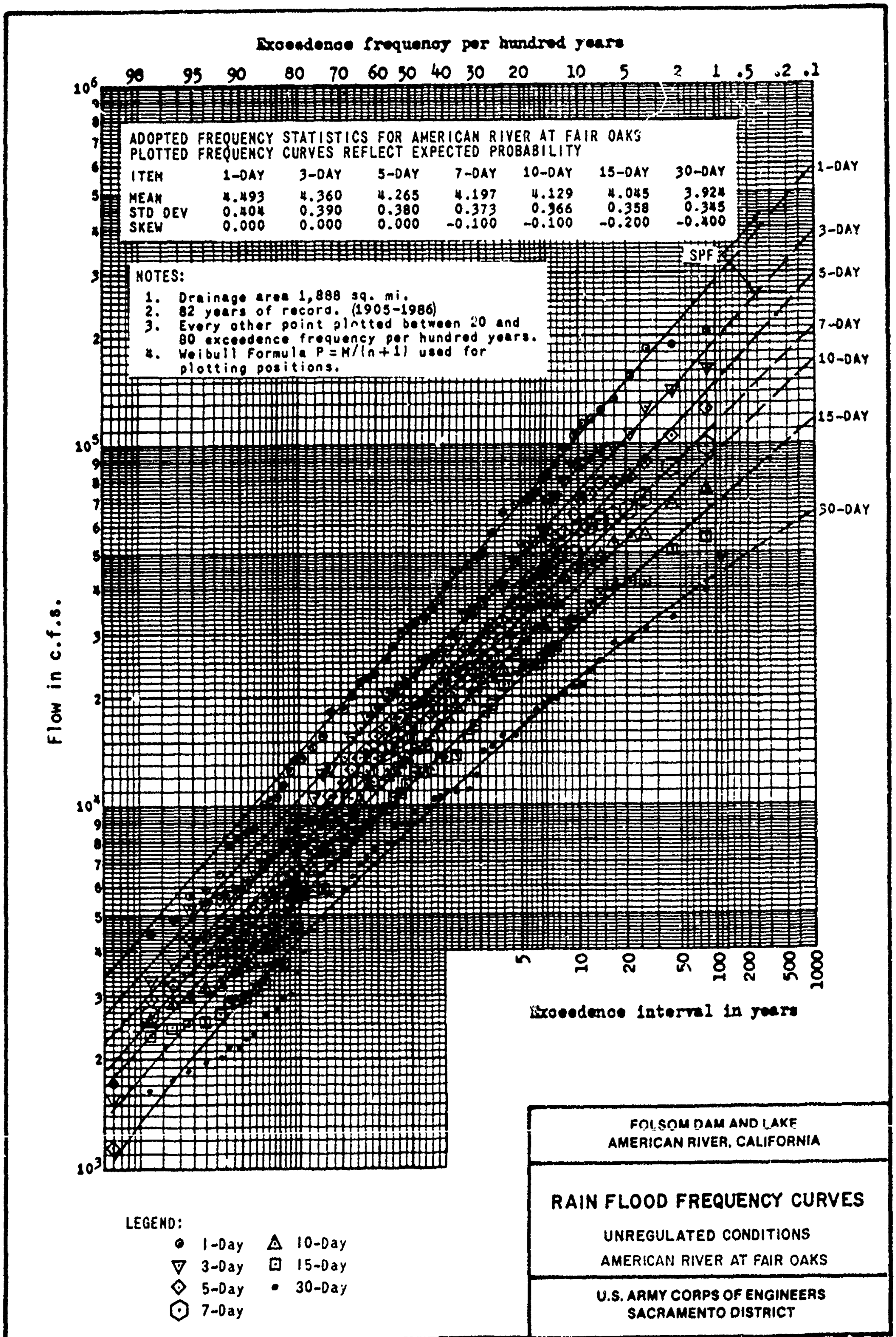
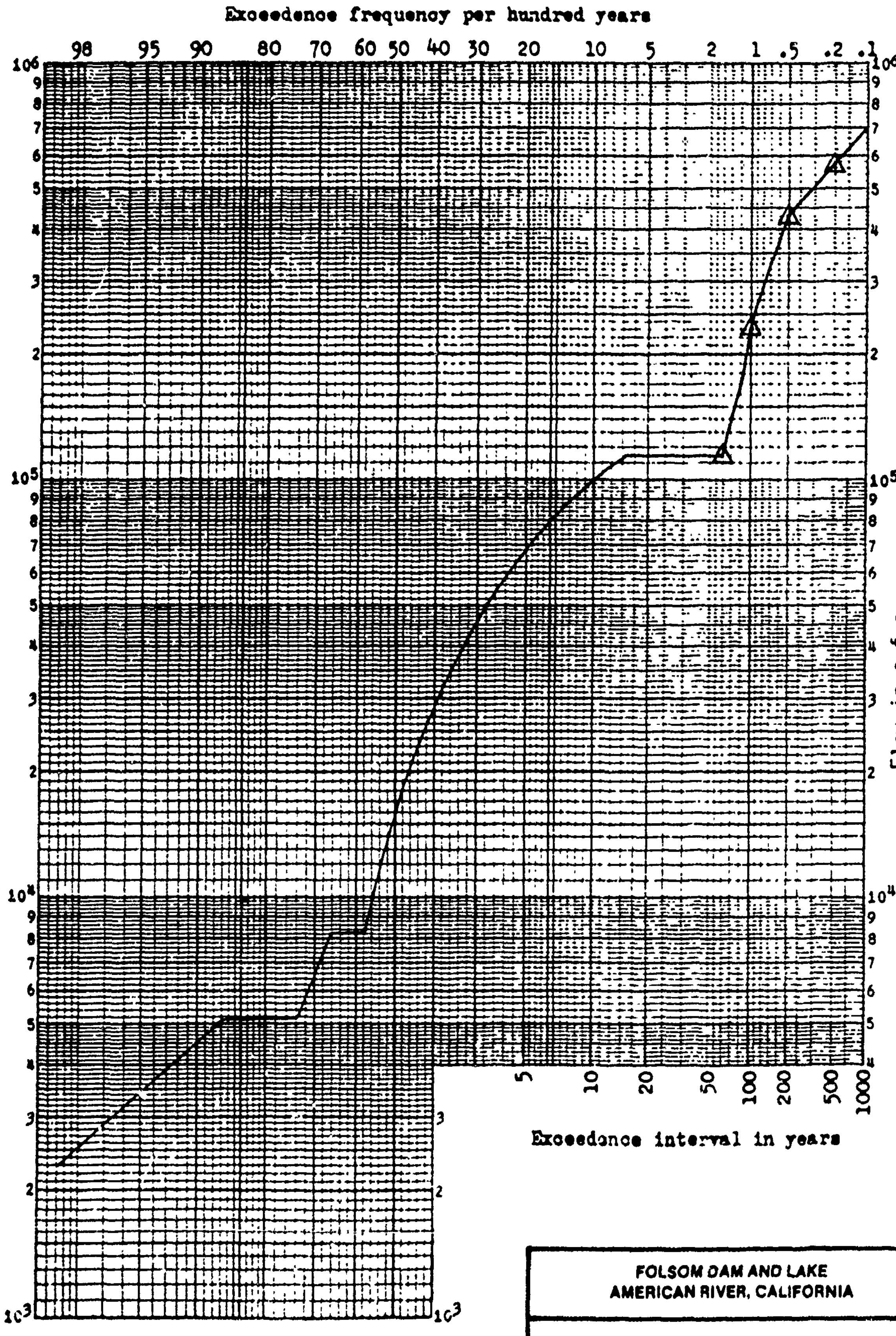

Exceedence frequency per hundred years
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2 .1
ADOPTED FREQUENCY STATISTICS FOR AMERICAN RIVER AT FAIR OAKS
PLOTTED FREQUENCY CURVES REFLECT EXPECTED PROBABILITY
ITEM 1-DAY 3-DAY 5-DAY 7-DAY 10-DAY 15-DAY 30-DAY
MEAN 4.493 4.360 4.265 4.197 4.129 4.045 3.924
STD DEV 0.404 0.390 0.380 0.373 0.366 0.358 0.345
SKEW 0.000 0.000 0.000 -0.100 -0.100 -0.200 -0.400
SPF
NOTES:
1. Drainage area 1,888 sq. mi.
2. 82 years of record. (1905-1986)
3. Every other point plotted between 20 and 80 exceedence frequency per hundred years.
4. Weibull Formula P=M/(n+1) used for plotting positions.
1-DAY
3-DAY
5-DAY
7-DAY
10-DAY
15-DAY
30-DAY
Flow in c.f.s.
5 10 20 50 100 200 500 1000
Exceedence interval in years
LEGEND:
1-Day
3-Day
5-Day
7-Day
10-Day
15-Day
30-Day
FOLSOM DAM AND LAKE
AMERICAN RIVER, CALIFORNIA
RAIN FLOOD FREQUENCY CURVES
UNREGULATED CONDITIONS
AMERICAN RIVER AT FAIR OAKS
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Figure 4



Notes:

1. The project curves, to the 50 year event, reflacts 32 years of record (1953-1986).
2. The remaining portion of the curve reflect the results of hypothetical flood routings as represented by the plotted points.
3. The hypothetical routings used the present authorized flood operation of Folsom Dam.

FOLSOM DAM AND LAKE
AMERICAN RIVER, CALIFORNIA

PEAK FLOW FREQUENCY CURVE

REGULATED CONDITIONS
AMERICAN RIVER AT FAIR OAKS

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

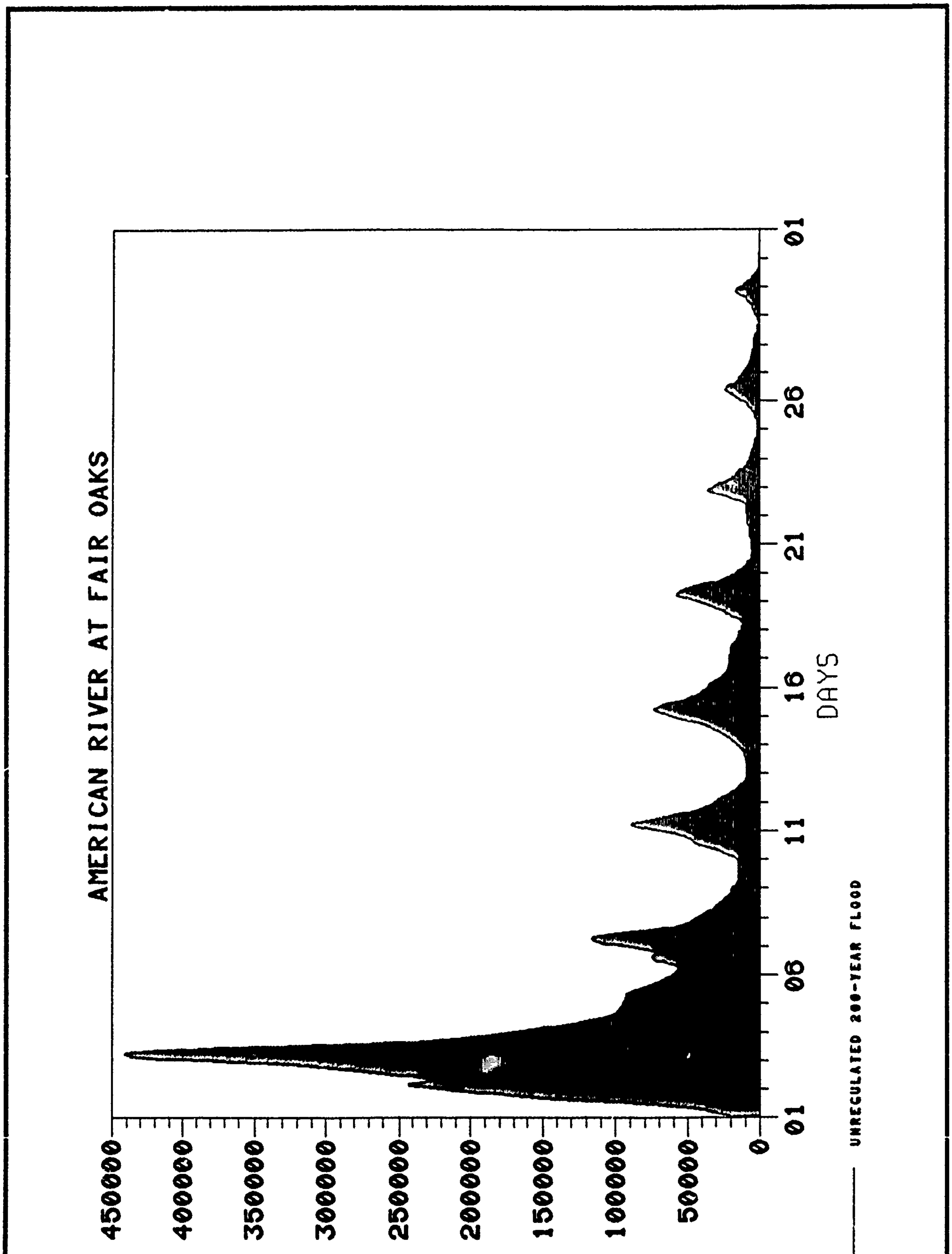

AMERICAN RIVER AT FAIR OAKS
FLOW IN CFS
450000
400000
350000
300000
250000
200000
150000
100000
50000
0
01
06
11
16
21
26
01
DAYS
UNREGULATED 200-YEAR FLOOD

Figure 6

hydroelectric power generation. Pertinent information about these upstream reservoirs is shown in Table 1. Historically, they have provided some incidental flood control benefits but operationally it would be very difficult to rely on this storage for the following reasons:

1) The upstream reservoirs control a maximum of only 18% of the runoff into Folsom.

2) Most of these reservoirs are high in the basin where much of the precipitation during major storms falls as snow.

3) The reservoirs have very limited outlet capacities and therefore may provide some protection until filled by the first major storm, but are of limited benefit during succeeding floods unless the outlets undergo major reconstruction; and

4) It may be unwise to allocate credit to reservoirs so high in the basin because their effectiveness is dependent on storm centering. For example, after the storm of February 1986, not all upstream reservoirs filled. The available space was therefore ineffective.

TABLE 1

PERTINENT INFORMATION FOR SEVERAL UPSTREAM RESERVOIRS

Reservoir	Drainage Area (sq-mi)	Capacity (ac-ft)	Distance to Folsom (river miles)
French Meadows	47	136,400	61
Hell Hole	114	207,600	68
Loon Lake	30	76,500	75
Union Valley	84	271,000	57
Ice House	27	46,000	64

Raising Folsom Dam. Raising the dam was not considered further because of the miles of dikes associated with the dam. The cost in raising the dikes and the marginal increase in flood protection precluded this from further study.

Additional flood space at Folsom. Increased flood space at Folsom, up to a maximum of 650,000 acre-feet, could raise flood protection to just under the 100-year level for an objective release of 115,000 cfs. Flood space greater than 650,000 acre-feet would severely affect other operational purposes of Folsom. In addition, a limited release capability at the lower pool elevations would offset much of the benefit of the increased space.

Lower Folsom Spillway. Lowering the spillway sill would increase the release capability by allowing dam releases to follow inflow as needed. As described earlier, Folsom is unable to release inflow early in an event until enough head is available to do so, at which point a significant amount of encroachment into the flood pool has occurred.

Increase Objective Release. Increasing Folsom objective outflows could also provide added flood protection on the lower American River. Objective outflows are based on the design and capacity of the levees and river channel downstream from the dam. Increased flows, thus, would require levee modifications at several locations downstream along the American River and tributary streams.

A summary of the affects of these measures individually and in combination are shown on Figure 7. Some combinations can provide up to a 150-year level of protection.

New Upstream Storage. Additional upstream storage is necessary to provide levels of protection in excess of 150-years. Previous studies have shown that the most practical location for a reservoir upstream from Folsom would be on the North Fork American River below the confluence of the North and Middle Forks near Auburn (see Figure 1). This location allows for a dam to be built to provide a storage capability large enough to significantly reduce downstream flooding. The basin above this site includes fifty-five percent of the total American River drainage, and historically has generated approximately two-thirds of the total runoff.

Conclusion

During the last 35 years, the American River has experienced several large floods near design magnitude. The existing flood control system was therefore evaluated to determine the level of protection provided for the Sacramento area. An analysis of the updated hydrology of the American River, unregulated and regulated flows, was performed to assist with the evaluation.

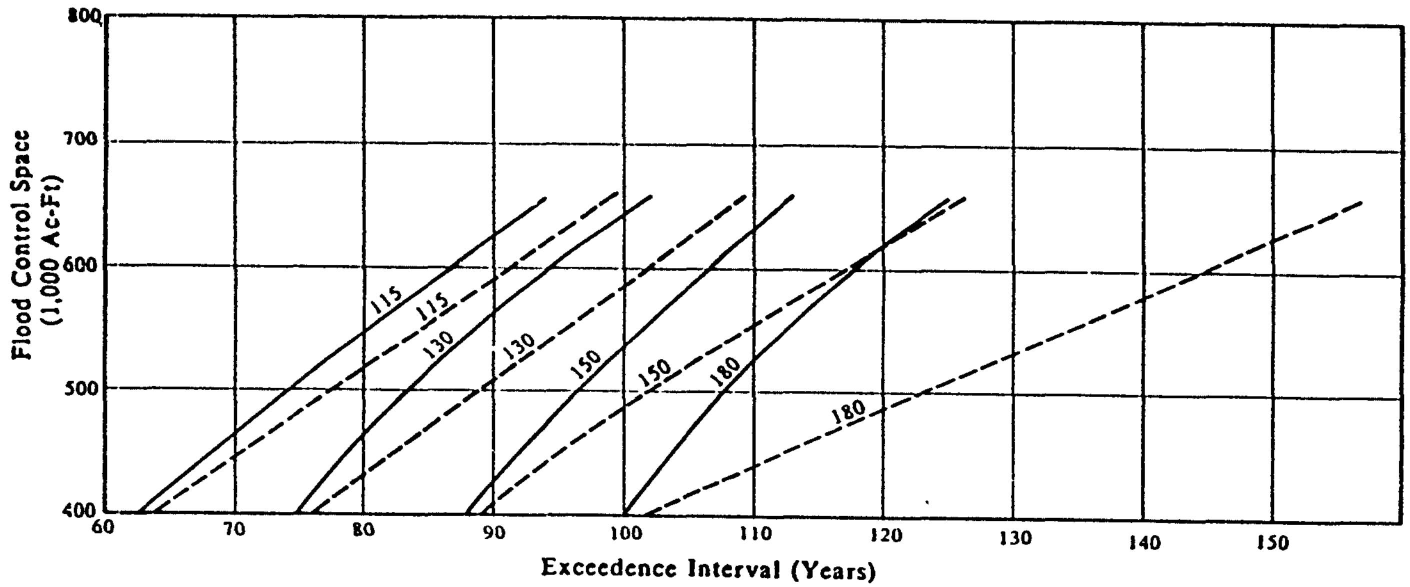
The analysis showed that Folsom Dam and downstream levees do not provide a high level of flood protection. To address this, several flood control measures were proposed which would enhance the existing flood control system. Each measure was evaluated on its own merits and in combination with other measures. These measures, excluding additional new upstream storage, could provide protection to a maximum level of 150-years. However, a high level of flood protection (i.e., about 200-years or greater) may be desirable for metropolitan areas, such as Sacramento, where levee failure could result in catastrophic loss of life and property. After extensive analysis, the construction of additional flood control space immediately upstream from Folsom Lake was found capable of effectively achieving the higher levels of protection along the mainstem American River.

References

1. Folsom Dam and Lake, American River, California, Water Control Manual, December 1987.

2. Special Study on the Lower American River, California, March 1987.

3. Bulletin #17B, "Guidelines for Determining Flood Flow Frequency", September 1981.



LEGEND:

115 Channel Capacity (1,000 cfs)
—— Existing Spillway (Maximum Pool Elevation 470.0)
---- Spillway lowered 15 feet (Maximum Pool Elevation 466.0)

NOTE:

80,000 Ac-Ft initial encroachment into flood space;
4 hr. delay on releases above starting release of 20,000 cfs;
Outflow = inflow lagged 4 hrs. until channel capacity is reached;
Maximum rate of change of release is 7,500 cfs/hr.;
- 47,000 Ac-Ft of upstream storage credit for exceedence intervals ≥ 100 yrs.

AMERICAN RIVER WATERSHED
INVESTIGATION, CALIFORNIA

FOLSOM DAM OPERATION
FOR
DOWNSTREAM CONTROL

SACRAMENTO DISTRICT, CORPS OF ENGINEERS
JANUARY 1988

Figure 7

Reevaluation of Frequency of Regulated Flows on the American River at Sacramento

by

Russell P. Yaworsky

SUMMARY OF DISCUSSION BY BRUCE C. BEACH

In response to a query, the author stated that the City and County were the local sponsors. The proposed Auburn Dam is quite controversial. The site is in another County, and local officials there oppose the dry dam alternative, they gain no benefits, but environmentalists oppose the permanent pool alternative.

Lew Smith, of OCE posed the question: Given the demonstrated uncertainty, is the use of a volume frequency curve the best way to determine Federal interest in a critical project like this? Mr. Yaworsky responded by asking the question: What alternatives are there?

FLOW REGULATION MODEL FOR THE PROPOSED HINGED POOL OPERATION OLMSTED LOCKS AND DAM OHIO RIVER

by

Lyndon C. Richardson, Jr.[1]

INTRODUCTION

The Olmsted Locks and Dam Project was authorized for construction by the Water Resource Development Act of 1988, which was approved in November 1988. The Olmsted Project will replace existing Locks and Dams 52 and 53 with a single project located 1.8 miles downstream of Locks and Dam 53 at Ohio River Mile (ORM) 964.4, near Olmsted, Illinois. The Olmsted Project is proposed to be operated as a "hinged pool." The hinge point for project operations is at Paducah, Kentucky located 30 miles upstream of the dam. The proposed hinged pool operation will require a more sophisticated flow regulation and pool control system than the "stair-step" operation now in use on the Ohio River Navigation system. This paper addresses the need for an unsteady flow regulation model for the proposed hinged pool operation at the Olmsted Project. A proposed unsteady flow regulation model is presented.

PHYSICAL SETTING AND AVAILABLE DATA

The Olmsted damsite is located on the Ohio River, 17 miles above the junction of the Mississippi River. Figure 1 is a location map of the project area. The project is the last of 19 modern high lift navigation structures to be constructed on the Ohio River system. These 19 projects replaced a system of 46 old low lift lock and dam structures, most of which had movable dams which were manipulated using semi-manual methods. The structures upstream of this project are the Smithland Locks and Dam at ORM 918.5, the Tennessee Valley Authority's (TVA) Kentucky Lock and Dam on the Tennessee River at mile 21.5 and Barkley Lock and Dam at Mile 27.6 of the Cumberland River. The Olmsted Locks and Dam is the most downstream navigation project on the Ohio River System. There are no navigation dams downstream of the project and none are planned. Open river navigation exists downstream of the project to the mouth of the Ohio River at Cairo, Illinois and thence to the Gulf of Mexico via the Mississippi River.

At the damsite, the Ohio River has a drainage area of about 203,000 square miles. The Tennessee and Cumberland Rivers contribute about 59,000 square miles which both join the Ohio River below Smithland Locks and Dam. The Mississippi River above Cairo, Illinois has a drainage area of 713,000 square miles. In spite of the 3.5 to 1 discrepancy in drainage area, the Ohio River contributes about 58% of the flow of the Mississippi River below the junction.

The stages of the Ohio River at the damsite are not uniquely related to the Ohio River Discharge but are a function of the flow coming down the Ohio River, the recent history of that flow, and the stage of the Mississippi River at Cairo, Illinois. Since the Mississippi and Ohio Rivers often respond to independent hydrologic stimuli, a wide variation in stage-discharge relations occur at

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[1]Hydraulic Engineer, Ohio River Division, U.S. Army Corps of Engineers

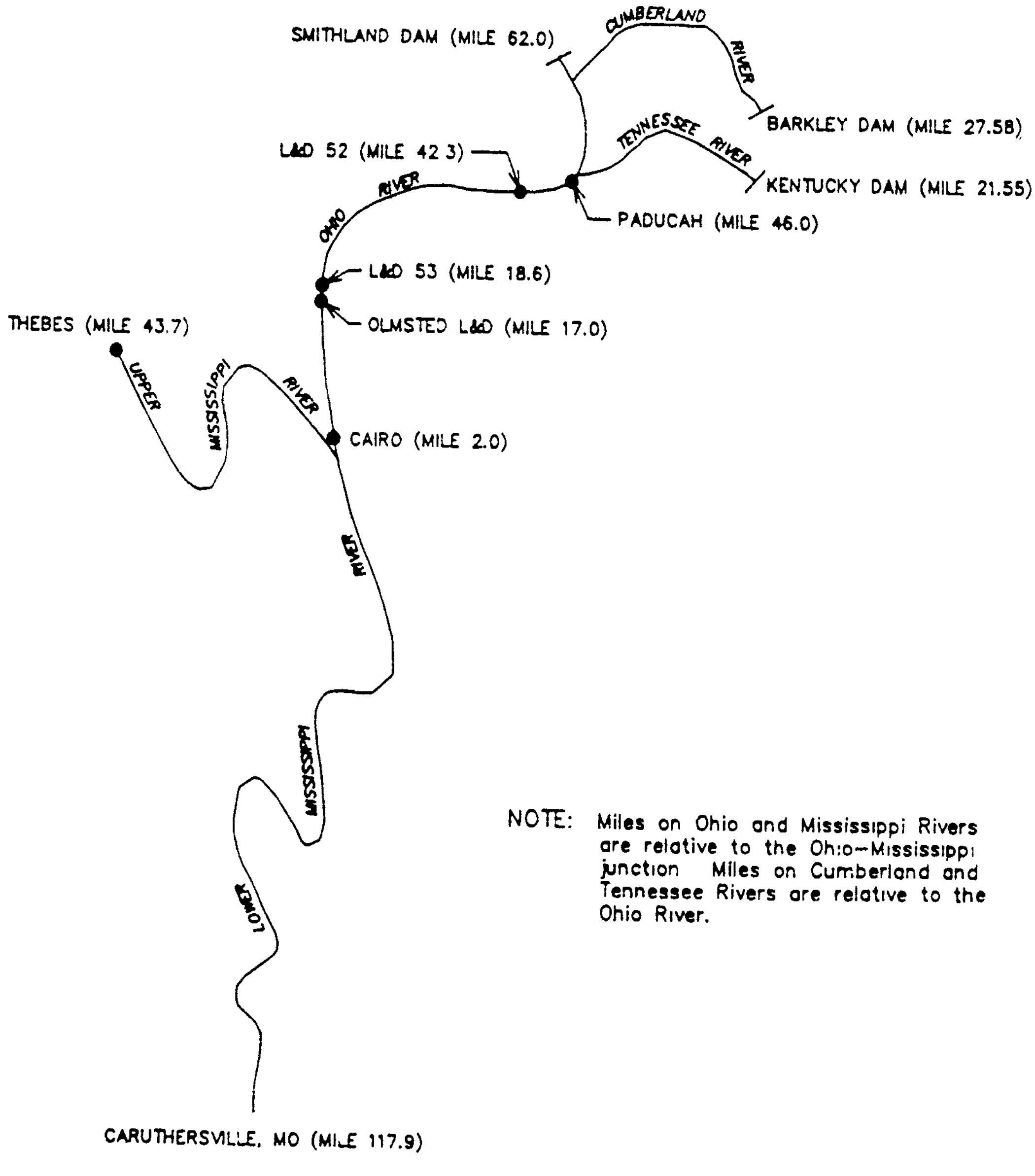


Figure 1. Map of study area

the damsite. Although a long historical record of stages and discharges is available for the area, they are not fully useful because of changed basin characteristics brought about by a wide variety of water resource developments over the past century. For example, over 80 major flood control dams have been constructed in the Ohio River basin during the last 60 years. These dams significantly affect flood flows and low flows on the Ohio River. Extensive systems of levees and floodways were constructed along the Lower Mississippi River. Over the years, these modifications have resulted in significant changes to the stage-discharge relations at the damsite.

For project design studies, it was necessary to select a data set which reasonably reflects the current hydrologic environment at the project site. It was necessary that this data set be appropriate to provide the required hydrologic engineering guidance relating to proposed project operations, risk analysis, navigation conditions and real estate acquisition. The data set selected for this purpose consists of a set of daily values of stage and discharge covering the period from October 1, 1966 through June 30, 1988. This period reasonably represents present day conditions because it represents the period after Lake Barkley was placed in service. Figure 2 is a plot of this data set and illustrates the wide variations in flow and stage that occur at the proposed damsite.

PROJECT DESCRIPTION

The Olmsted Project will feature twin 110-foot by 1,200 foot locks adjacent to the Illinois bank, and a 2,200 foot wide navigable pass dam extending from the locks to the Kentucky bank. There will be a total of 220 individually operated hydraulic wicket gates, each having a 10-foot nominal width with 4 inch gaps between them. The wickets will be 26 feet in length in the down position. Figure 3 is a plan of the proposed project.

HINGED POOL OPERATIONS

The 2,200-foot wicket gate dam will serve both as a navigable pass and as a flow regulatory section for the hinged pool operation. The hinge point for the Olmsted Project operation is the Paducah, Kentucky gage at ORM 934.6. The operational objectives of the proposed hinged pool as presented in the Supplement to the project General Design Memorandum (Louisville District, Corps of Engineers, 1990) are to:

1) Maximize open river (non-locking) time.

2) Maintain a minimum pool elevation of 300.0 feet at Paducah, Kentucky and elevation 302 feet at Smithland Lock and Dam tailwater.

3) Operate the dam efficiently by minimizing wicket gate operations.

4) Minimize hydraulic pulses and surges in the upper and lower pools by smoothly merging these pools during transitions from locking to open river conditions, and vice-versa.

Elevation 300 feet will be maintained at the hinge point except for instances of unusually low flows when slightly higher elevations will be maintained. This operational objective will maintain an adequate depth in the Paducah fleeting area, and maintain an adequate tailwater on the lower lock sill at the Kentucky Dam. A secondary operation objective will be to maintain elevation 302

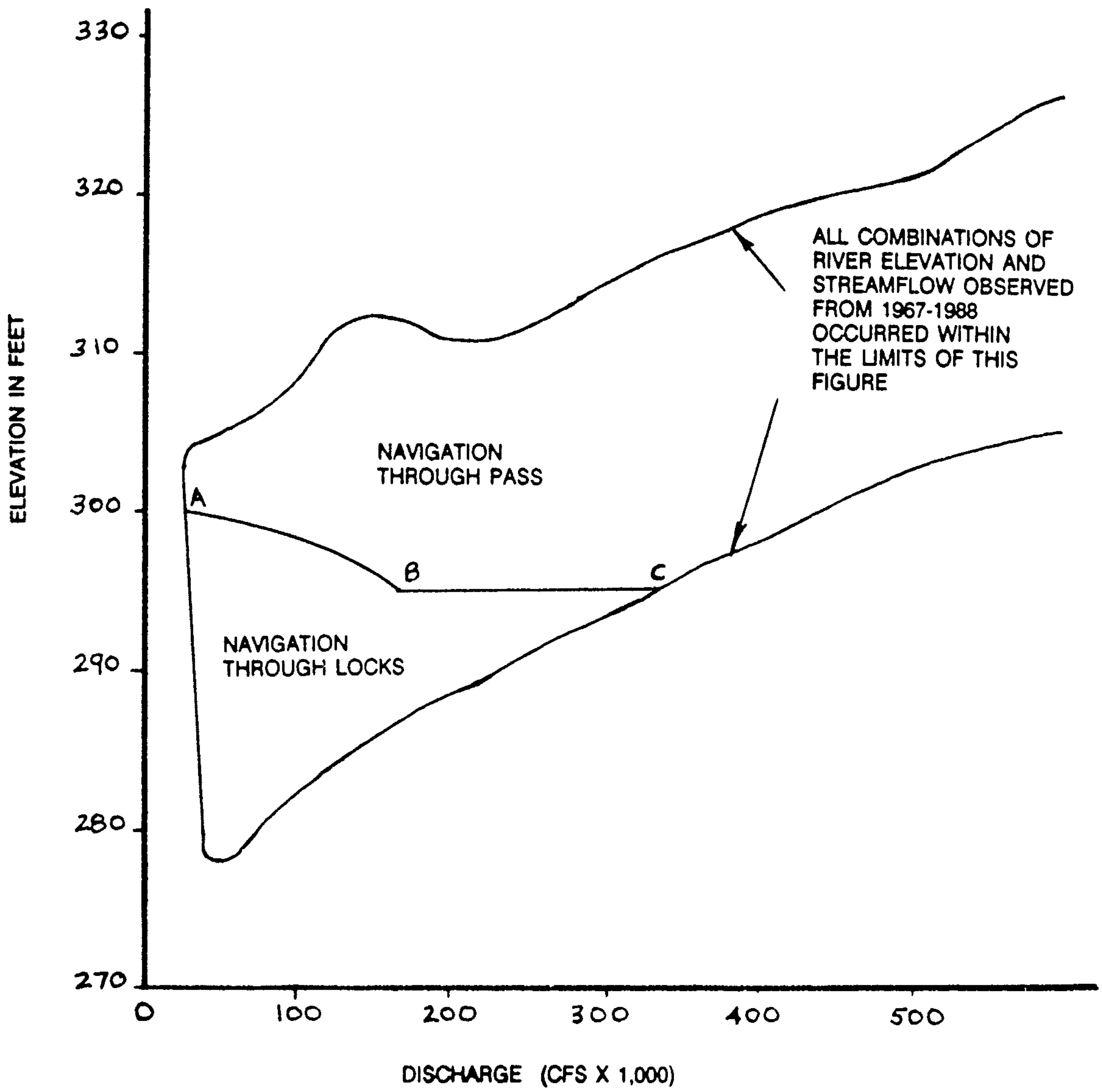


Figure 2. Olmsted Experience Curve -- Open River Navigation verses Locking

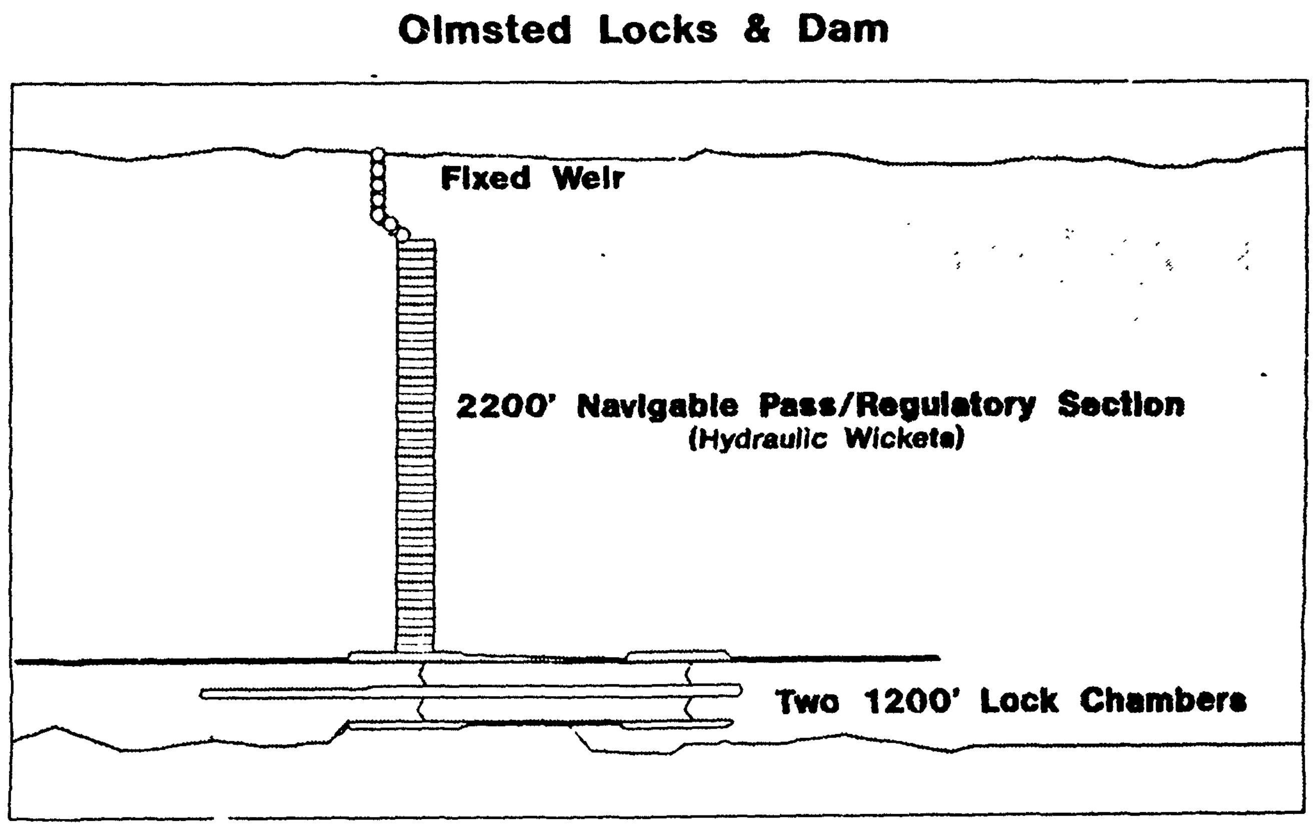


Figure 3. Plan of Proposed Olmsted Locks and Dam

feet at the tailwater of Smithland Locks and Dam which will assure adequate navigation conditions on the Cumberland River below Barkley Lock and Dam. This secondary objective will only be of interest during those periods of unusually low flow when the maintenance of elevation 300 feet at Paducah will not provide a tailwater elevation of 302 feet at Smithland. The hinged pool operation was chosen because of the following reasons. First, the operating pool levels will be kept as low as possible to minimize any potential adverse increase in groundwater stages in the environmentally sensitive wildlife areas upstream of the damsite. Second, use of the hinged pool maximizes the time "open river" navigation is possible during which commercial navigation can bypass the locks and can transit the lowered navigation pass section. This open river navigation provides significant savings in time and costs for the waterway users because it eliminates the delays and added fuel costs associated with locking. It also enables the use of larger barge tow configurations such as the 30 barge tows commonly used on the Mississippi River which cannot be accommodated by the 1,200-foot Ohio River locks without double locking. The hinged pool operation accomplishes this by smoothly merging the upper and lower pools together at elevation 295 feet which provides satisfactory water depth above the navigation pass sill.

For this discussion, the term "open river" shall be used to describe conditions when navigation can bypass the locks and can transit the lowered pass section. Figure 2 graphically displays this open river time. The demarcation between the time navigation occurs through the locks and when it occurs through the navigable pass is shown by Line 'A-B-C' of Figure 2. Any time the stage-discharge relationship falls below and to the left of Line 'A-B-C', the locks will be in service and the navigable pass will be closed to traffic. If the stage-discharge relationship falls above and to the right of Line 'A-B-C', then the project will be in the open river status with all wickets lowered. The locks will be out of service and the navigable pass in service. It is apparent that whenever the tailwater elevation at the dam is above elevation 300 feet, the navigable pass wickets will be in the lowered position and all navigation traffic will use the navigable pass. Also, because of the configuration of the sill of the navigable pass, the navigable pass cannot be opened to traffic until the tailwater elevation at the dam has risen above elevation 295 feet. This insures a minimum satisfactory depth of 15 feet across the sill of the pass section of the dam. During these periods, the navigable pass wickets will be in the raised position and the navigable pass will be closed to traffic.

During locking, the wickets will be raised and lowered as required to hold the upper pool as low as possible while maintaining elevation 300 feet at the hinge point. The upper pool will not be drawn down below elevation 295 feet. The upper pool will be maintained by the wickets with these objectives and, during the transition period from locking to open river, will also be operated to minimize project swellhead by smoothly merging the upper pool and the tailwater. There will be occasional short periods when the tailwater will rise above elevation 295 feet because of backwater from the Mississippi River, but, because of insufficient flow on the Ohio River, the navigable pass cannot be lowered because the proper pool elevation cannot be maintained at Paducah. This case is illustrated on Figure 2 by the curved portion on the left end of the demarcation line 'A-B-C' between locking and open river. During locking periods, the normal operating range for the upper pool will be between elevations 295 and 300 feet. However, during periods of very low flow on the Ohio River, the upper pool must be maintained at elevations between 300-301.5 feet in order to provide a minimum tailwater elevation of 302 feet at the Smithland Locks and Dam, located 47 miles upstream.

The navigable pass will be open about 59 percent of the time, although this will vary from year to year. For example, in low flow years, open river navigation might only occur for 30 percent of the time, but might occur as much as 80 percent of the time in a "wet" year. Locking is most common in the period from early fall through mid-January. Open river conditions occur most often from mid-winter through late summer. In a typical year the navigable pass is lowered and raised 5-7 times.

TYPICAL OPERATING SCENARIOS

Case One -- Transition From Locking to Open River Navigation. Figure 4 illustrates the proposed hinged pool operation of the Olmsted Project for the conditions of a rising river. In this example, the rise in project tailwater is assumed to be influenced mainly by increasing Ohio River discharge. Backwater caused by the Lower Mississippi River is assumed to be negligible. For this case, the tailwater level is assumed to rise on the curve labeled, "Design Critical Tailwater." Initially, the navigable pass is assumed to be in the raised position with open river navigation suspended, all navigation traffic is locking and the initial headwater elevation is at elevation 300 feet (Point "A" in Figure 4). As the discharge increases, the project headwater is brought down on curve A-B by progressively lowering wickets in such a manner as to maintain elevation 300 feet at Paducah. The headwater elevation is held at elevation 295 feet by the wicket gates until one foot of swellhead across the dam is reached (Point "C" in Figure 4). From this point, a swellhead of one foot is maintained until the tailwater elevation reaches 295 feet (Point "D" in Figure 4). At this point, the navigable pass would be completely lowered and open river navigation would commence. At this point, the project would exert no influence on river stages upstream of the project.

Case Two -- Transition From Open River Navigation to Locking. For conditions of a falling river, the navigable pass wickets will be raised in reverse as in the foregoing. The navigable pass will be closed when the river flow and tailwater elevation are insufficient to maintain the minimum navigable depth across the dam sill (tailwater elevation 295 feet) and to maintain elevation 300 feet at Paducah. During the transition from open river navigation to locking, the navigable pass must be raised gradually to prevent the formation of undesirable transitory waves upstream and downstream of the dam. In the past, rapid raising of the wickets at existing Locks and Dam 53 resulted in the formation of a downstream trough or negative wave. These waves adversely affect navigation on the Lower Ohio River and the Lower Mississippi River.

FLOW REGULATION REQUIREMENTS

The hinged pool operation will require a more sophisticated operating system than is now in use on the Ohio River Navigation system. This is because the project gate settings must be made earlier than actual flow conditions in order to allow for the hydrodynamic lag between gate operations and their later effect at the hinge point. In order to establish at any given time the required settings for the wicket gates to meet the foregoing operating objectives, the operating system for flow regulation must include knowledge of the hydraulic state of the pool. In this application, the instantaneous state of the pool is characterized by the volume. The flow regulation system must also be capable of predicting what the anticipated future state of the pool will be. Hence, the flow regulation system must be capable of processing data on inflow from the Ohio River and tributaries. Here, simple reliance on the headwater elevation of the pool to ascertain the pool state will not be possible because (1) the free surface of the pool is not horizontal and (2) the flow regime is unsteady because of hydropower releases from the Barkley and Kentucky Dams and because of changes in gate settings at the Smithland Locks and Dam. Under these circumstances, knowledge of the headwater elevation of the pool at Olmsted Dam will provide only incomplete, partial indications of the pool state, insufficient for operating the proposed hinged pool. It should be noted that these requirements are only of concern during locking periods at the Olmsted Project. Whenever the flow at Olmsted exceeds 270,000 cfs, flow

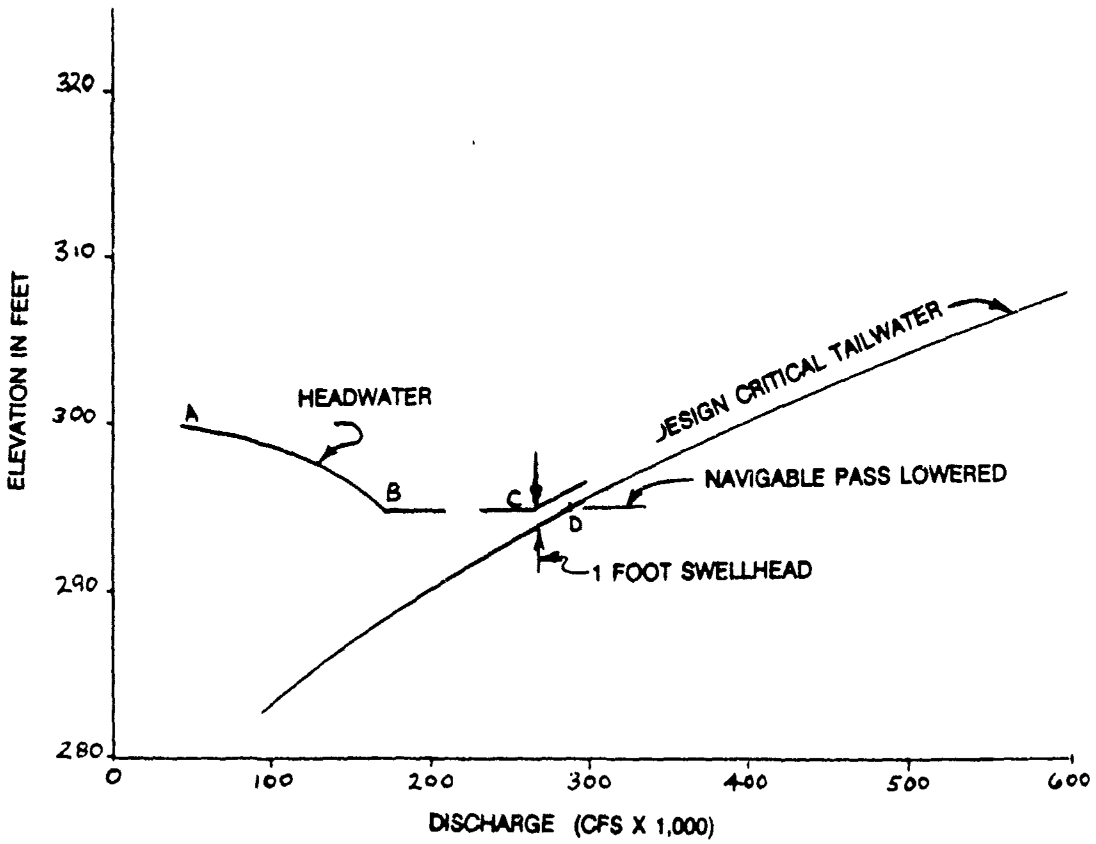


Figure 4. Operating Scenario -- Transition From Locking to Open River Navigation

regulation will cease because the navigable pass will be open. However, during periods of lesser flow when the navigable pass wickets are used to control the pool or when the transition from open river navigation to locking operations is being accomplished, pool control will be more complex.

AUTOMATED POOL CONTROL

Eventually, a total project operating system is envisaged for the Olmsted Locks and Dam in which most of the gate controls will be automated and driven by a master control system. A plan of such an automated flow regulation system is shown schematically in Figure 5. This automated system would be used to regulate flow past the dam any time the project is not in the open river status. The automated system would consist of four components or modules. These components consist of an operation control module, a data acquisition module, a numerical model, and a flow conversion and gate adjustment module. The function of each of these components is summarized below.

Operations Control Module. The operations control module would independently carry-out the decision process for Olmsted Locks and Dam. These decisions would be based on the current and future system states, operational rules, and objectives. The operations control module would function with the aid of an externally maintained information base. Included in this information base would be both operation rules (e.g., maximum rate of change of flow at Olmsted) and the operational objectives (e.g., fixed pool elevation at Paducah, Kentucky). Also included within the knowledge base would be the tolerance limits for each of the objectives and operating rules.

Data Acquisition Module. Real-time data will be used to operate the model and will include both flows and stages. The acquisition and screening of this real-time data would be the responsibility of the data acquisition module. The flows that will be required will be the Smithland Lock and Dam discharge, the Barkley Dam outflow, the Kentucky Dam outflow, ungaged local inflows, and the flows from the Upper Mississippi River. The stages that will be required include the stage at the upstream hinge point at Paducah, Kentucky, the stage at Smithland Locks and Dam tailwater, and the headwater and tailwater elevations at the Olmsted Dam. It may be necessary to acquire additional stage information at other points within the pool to determine the state of the pool. Estimates of future (forecast) inflows from Smithland Locks and Dam, Barkley Dam, and Kentucky Dam and the upper Mississippi River will be required.

Mathematical Model. Determination of the current and future hydraulic states would be made by a numerical model. The model proposed for this function is the current FLOWSED one-dimensional numerical model for computing unsteady flows on the Ohio River and its major tributaries. Part of the requirement for using this model for real-time operation is the need for an algorithm to handle the hinged pool operation. After a discussion of the flow conversion and gate adjustments module, the remainder of this paper will address the FLOWSED numerical model and the modifications made to the model to handle the hinged pool operation.

Flow Conversion and Gate Adjustment Module. The module used to determine the Olmsted gate settings would require information on the future flows to be released from Olmsted. These flows will be computed by the numerical model using real-time conditions and operations determined by the operations control module. The current project headwater and tailwater elevations will be required. The current wicket gate settings and the operational status of the wickets will be required. The position of the wickets would be monitored continuously by an industrial quality microprocessor gate monitoring and control system. These wicket settings would

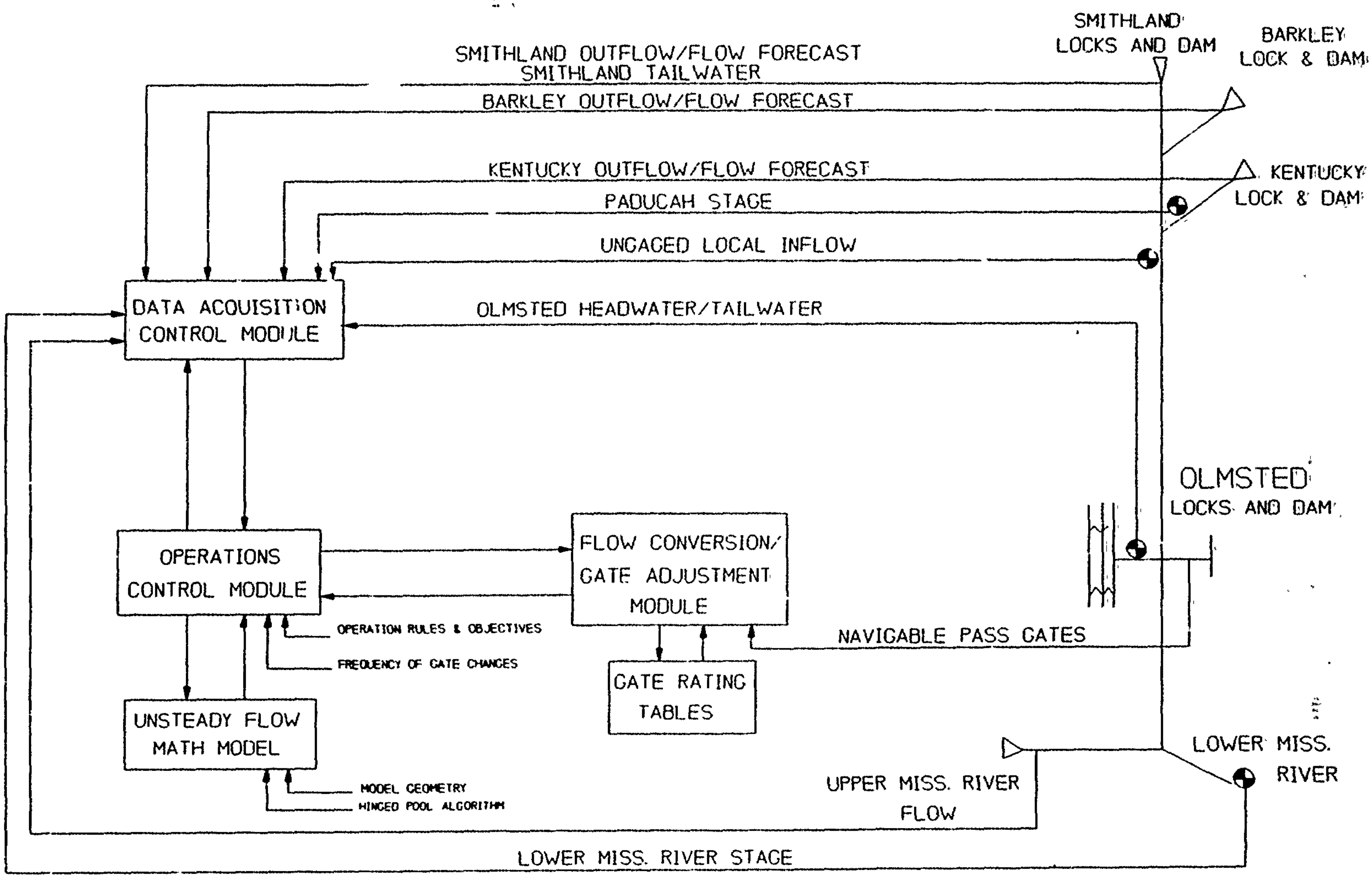


Figure 5. Proposed Automated Flow Regulation System for Olmsted Locks and Dam

be displayed both digitally and graphically on video display consoles in the main locks and dam operations control center. The Lock operation personnel will always have a graphical representation of the configuration of the navigable pass. Once the required wicket gate settings have been determined, the module would display these changes and if desired, transmit the required instructions to the gate operating machinery in the dam via a programmable logic controller. For safety reasons, the capability of manual override of the automated wicket operating system would be available through hardwired controls.

FLOWSED -- UNSTEADY FLOW NUMERICAL MODEL

Summary of Model Capabilities. The original FLOWSED model was developed at the Colorado State University (Chen, 1973). Through funding by the Ohio River Division, the FLOWSED model was modified by the Waterways Experiment Station (Johnson, 1982) to account for the effect of the navigation locks and dams on the Ohio River. The program was subsequently restructured and the sediment computations removed from the program by the Ohio River Division but the name FLOWSED has been retained. A complete, theoretical discussion of FLOWSED would take more space than is appropriate here. A thorough discussion of the background and theory of the program is given by Johnson (Johnson, 1982). Only an overview of the program capabilities is given here. FLOWSED is a 1-Dimensional, unsteady flow, implicit finite difference model that provides the capability of dynamically modeling a system containing any number of tributaries. FLOWSED has the special capability to model the influence of high-lift navigation dams in the system. This special locks and dams feature is a key feature of the program for modeling unsteady flows on the Ohio River system because of the many high-lift locks and dams on the Ohio River navigation system.

Channel Geometry. The channel geometry is modeled by input of tables of elevation versus flow area, topwidth, and Manning's n-values at each cross section (computation point) along the study reach. The n-values are allowed to vary with elevation at a particular cross section and with distance along the channel. The channel geometry used for this study is shown in Figure 1 and includes the reach of the Ohio River below Smithland Locks and Dam, the Cumberland River below Barkley Dam, the Tennessee River below Kentucky Dam, and a portion of the Mississippi River above and below the confluence of the Ohio River at Cairo, Illinois. Cross sections are spaced at approximate 1 mile intervals along the Ohio River and tributaries and at 5 mile intervals along the Mississippi River. Additional cross sections are provided at locations of locks and dams and at gaging stations.

Model Boundary Conditions. The upstream boundary conditions are prescribed by outflow discharge hydrographs at Smithland Locks and Dam, Barkley Dam, Kentucky Dam and the Upper Mississippi River discharge at Thebes, Missouri. The downstream boundary for the model is a discharge rating curve for the gaging station at Caruthersville, Mississippi on the Mississippi River. Ungaged local inflows are treated as lateral inflows input into the model at the appropriate locations.

Initial Conditions and Time Step. Initial conditions can be specified by input of a steady flow water surface profile with elevation and flow at each cross section or by a transient profile from previous computations. The solution becomes independent of the initial conditions after a sufficient length of time. A 1-hour time step for computations has been found to yield satisfactory results for model applications on the Ohio River and tributaries.

Locks and Dams. The FLOWSED model treats locks and dams as discontinuities or internal boundaries, wherein there is an elevation change across the dam with no change in discharge. Two methods can be used to specify the way in which the dam is to be operated. The normal procedure for handling locks and dams is to input constant elevations upstream of a lock and dam to reflect the pool elevation the lock operator is expected to maintain. With this procedure, FLOWSED computes the flow required to be passed through the structure in order to maintain the upstream pool elevation required. Theoretically, the operator could use the gate rating tables to make the gate adjustments required to pass the computed flow. Alternatively, a time-varying upper pool stage hydrograph can be specified as an interior boundary instead of a constant stage to be maintained. The proposed hinged pool operation at Olmsted Locks and Dam is a special case of the latter and requires special treatment as discussed next.

Hinged Pool Algorithm. An experimental algorithm for modeling the hinged pool operation was added to the FLOWSED model (Johnson and Weisinger, 1990). The algorithm is based upon prescribing a time-varying upper pool stage hydrograph at Olmsted but the prescribed elevation is determined in a different manner. The hinged pool algorithm is based upon the use of results from several steady flow runs in which various combinations of inflows and Olmsted elevation settings were prescribed to determine the corresponding water surface elevation at Paducah and downstream of Smithland Locks and Dam. For example, it is known that if the sum of the steady flow discharges from the Smithland and Barkley dams is less than 65,000 cfs, the Olmsted elevation must be prescribed to be 300 feet to force the Smithland tailwater above elevation 302 feet. The algorithm uses three steps to prescribe the water surface elevation upstream of the Olmsted Dam. First, the algorithm computes elevation settings at Olmsted to force the tailwater elevation at Smithland above 302 feet. Next, the program checks to determine if the Smithland tailwater from the previous time step is greater than 302 feet. If so, then an Olmsted headwater elevation that will force the elevation at Paducah to remain near elevation 300 feet is computed and used. The computed hourly elevations at Olmsted are saved and at the end of an operational cycle are smoothed using a three-point moving average equation, i.e., the past, present and future elevations are averaged. After the elevation hydrograph is smoothed, the complete flow regulation cycle is rerun with the smoothed elevations prescribed as the time-varying boundary condition upstream of the dam. This smoothing technique is required to prevent the elevation hydrograph and the discharge through the dam from becoming to erratic and causing excessive number of wicket gate operations. Several flow events were simulated to verify the model and the behavior of the Olmsted hinged pool algorithm. Results of some of these are described next.

Simulation of May-June 1988 Low Flow Period -- Without Olmsted Project. Previous applications of FLOWSED on the Ohio River were primarily concerned with modeling flood flows. Since low flows are of primary interest in this application, it was believed necessary to select a recent low flow period to verify the model performance, geometry, and roughness, without the Olmsted Project inplace. The low flow period from 20 May 1988 to 10 June 1988 was selected for this application. For this application, existing Locks and Dams 52 and 53 were left in the model as internal boundaries and the recorded headwater elevations for the entire period at these projects were prescribed as input. The upstream boundary conditions were input as the observed discharge hydrographs for the Smithland Dam, Barkley Dam, and Kentucky Dam and the observed flow hydrograph for the Upper Mississippi River at Thebes, Missouri. The results in the form of elevation plots are presented in Figures 6 and 7. These figures show comparisons of the computed and recorded stage hydrographs at Paducah, Kentucky and at the Smithland Locks and Dam tailwater. These results are considered satisfactory at this stage of study and compare within 0.5 to 1.0 feet.

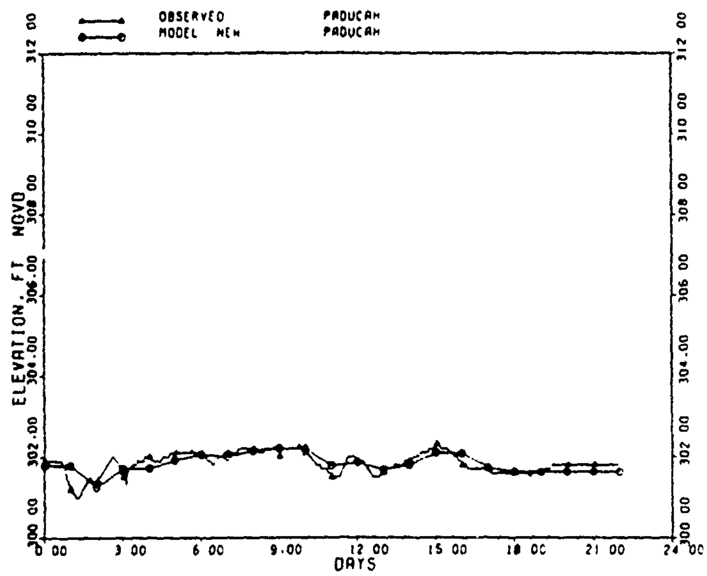


Figure 6. Comparison of Observed and Computed Elevations at Paducah, Kentucky for May - June 1988 Without Olmsted Inplace

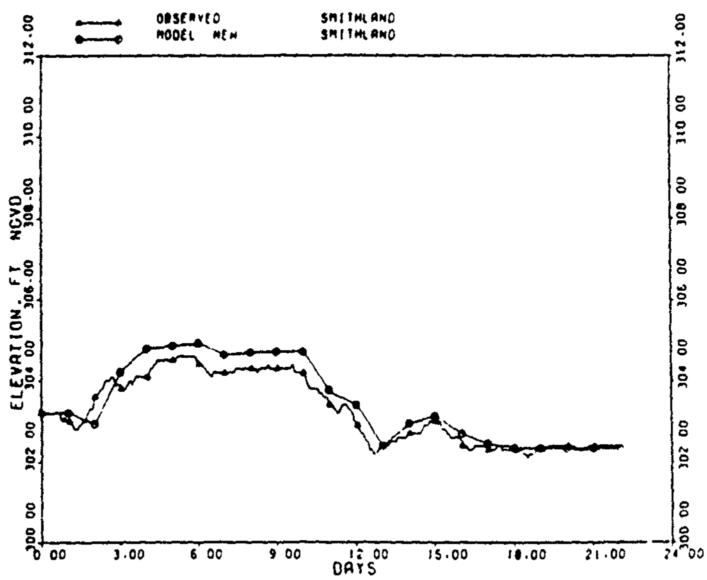


Figure 7. Comparison of Observed and Computed Elevations at Smithland Dam for May - June 1988 Without Olmsted in Place

Simulation of May-June 1988 Low Flow Period -- With Olmsted Project Inplace. The FLOWSED model was modified by replacing Locks and Dams 52 and 53 in the model with the Olmsted Project. The hinged pool algorithm was introduced into the computer code as a separate subroutine. The May-June 1988 flow period was simulated with Olmsted inplace. As can be seen in Figure 8, the Paducah, Kentucky elevation is maintained throughout the simulation near 300 feet; however, Figure 9 shows that the Smithland and Barkley outflows are not sufficient the last 7 days to maintain the tailwater at Smithland above elevation 302 feet. During this period, Figure 10, shows that the Olmsted Project is exercising maximum control of the pool; i.e., an elevation of 300 feet is being forced at the dam. Under actual conditions, it is envisioned that the maximum pool elevation at Olmsted would be allowed to rise above elevation 300 feet for very short and infrequent periods in order to hold elevation 302 feet at Smithland. The capability to retain pool levels above elevation 300 feet at Olmsted will depend on the amount of flow available, lockage water requirements, gate leakage and the maximum damming height of the wickets.

With Locks and Dams 52 and 53 removed from the system and the Olmsted Dam inplace, several hypothetical inflow events were simulated to demonstrate the behavior of the Olmsted hinged pool algorithm. The results of two of these simulations are summarized below.

Simulation of Smithland Dam Flow Event. For this event the Barkley and Kentucky Dam outflows were held constant at 6,000 and 12,000 cfs, respectively while the Upper Mississippi River flow was taken to be a constant 120,000 cfs. As illustrated in Figure 11, the Smithland dam outflow had the flow increasing from 40,000 cfs to 200,000 cfs over 6 days and then held constant for 3 days. Over the next 3 days the flow was decreased to 40,000 cfs and was again held constant for 3 days. Computed elevations at several locations are presented in Figures 12-14. As illustrated in Figure 12, Olmsted loses control from about day 9 to day 13. During this period all wicket gates would be lowered because the tailwater and headwater elevations have been merged. Open river navigation would occur through the navigable pass. From Figure 13, it can be seen that the elevation at Paducah, Kentucky can no longer be controlled and rises to a maximum of about elevation 302 feet during this period. Figure 14 shows that the tailwater at Smithland Dam rises to a maximum of about elevation 311 feet during this period. This simulation shows that the hinged pool algorithm functions properly throughout a flow event in which control is lost and then regained at Olmsted Dam.

Simulation of Barkley Dam Flow Event. For this simulation the Smithland Dam discharge was held at a constant 40,000 cfs along with 12,000 cfs and 120,000 cfs at Kentucky Dam and the Upper Mississippi River, respectively. The Barkley Dam outflow contains three rapidly varying flows in the first 5 days of the total 10 day simulation. These inflows are shown in Figure 15. As can be seen, the maximum flows for the three peaks are 20,000 cfs, 40,000 cfs and 60,000 cfs which simulate hydropower peaking operations at the Dam. Computed elevations at several locations are presented in Figures 16-18. As can be seen in Figure 16, the peak of the maximum surge created at Barkley Dam is about 16 feet. From Figure 17, it can be seen that operation of the Olmsted Dam hinged pool attenuates the surge at Paducah, Kentucky. The surge shows up again downstream of Olmsted Dam as shown in Figure 18.

CONCLUSIONS

Discussion. This paper addresses the need for an unsteady flow regulation model for the Olmsted Locks and Dam Project. The current numerical model called FLOWSED for computing unsteady flows on the Ohio River navigation system was modified to handle the proposed Olmsted Project. These modifications were required since the Olmsted pool is to be operated as a "hinged

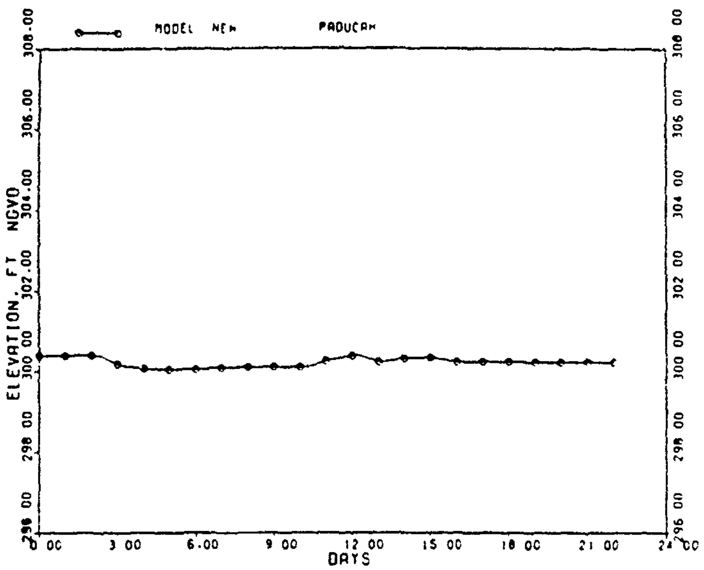


Figure 8. Computed Elevations at Paducah, Kentucky
for May - June 1988 With Olmsted Inplace

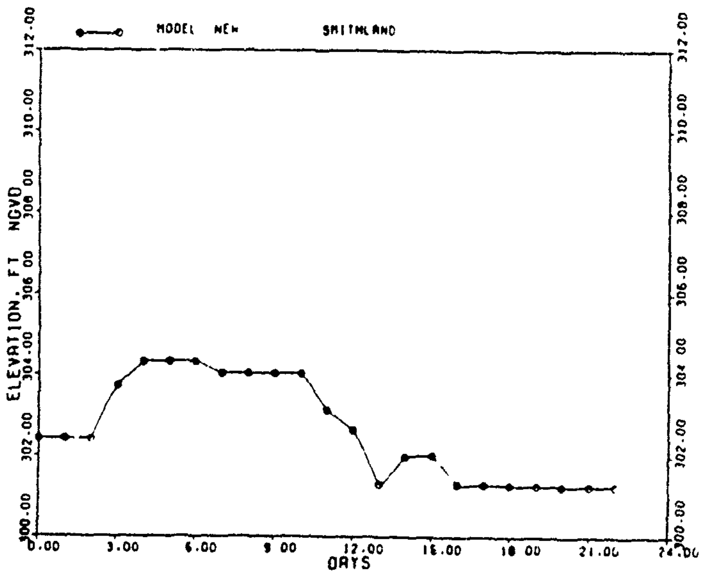


Figure 9. Computed Smithland Tailwater for
May - June 1988 With Olmsted Inplace

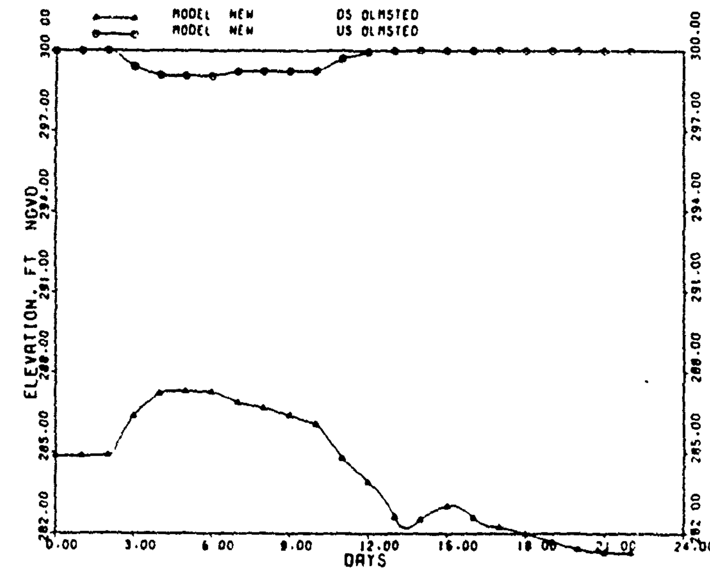


Figure 10. Computed Olmsted Headwater and Tailwater for May - June 1988 with Olmsted Inplace

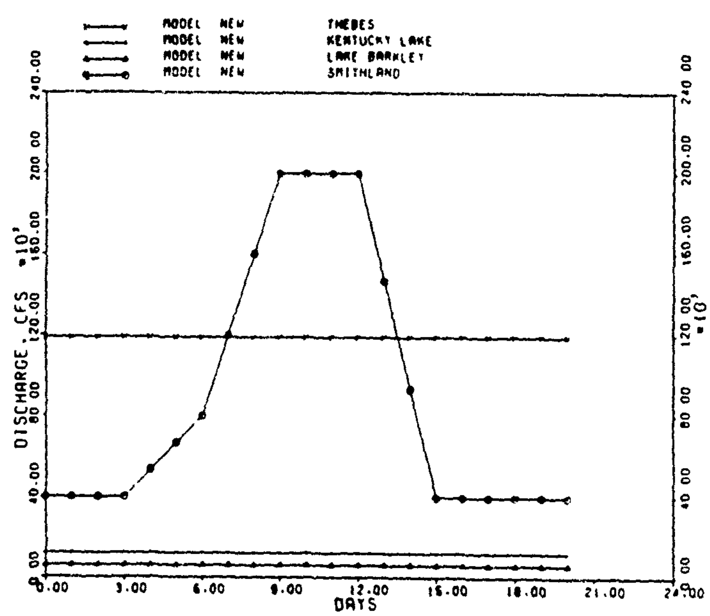


Figure 11. Inflows for Smithland Flow Event

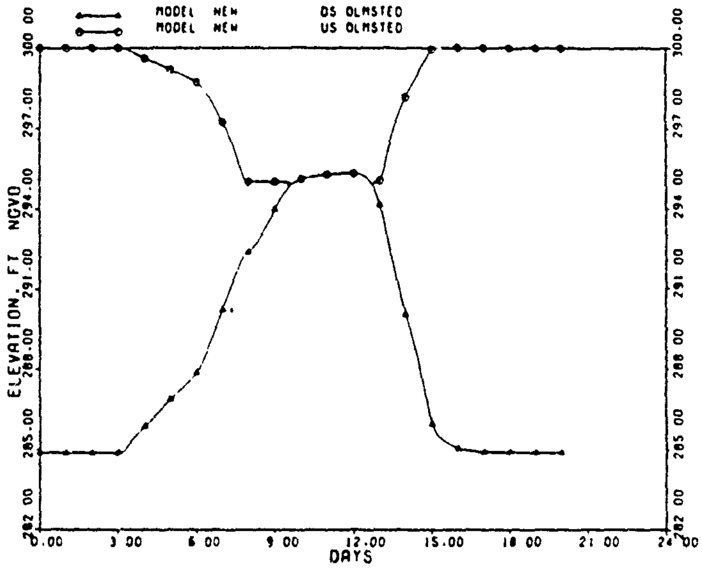


Figure 12. Computed Olmsted Headwater and Tailwater for Smithland Flow Event

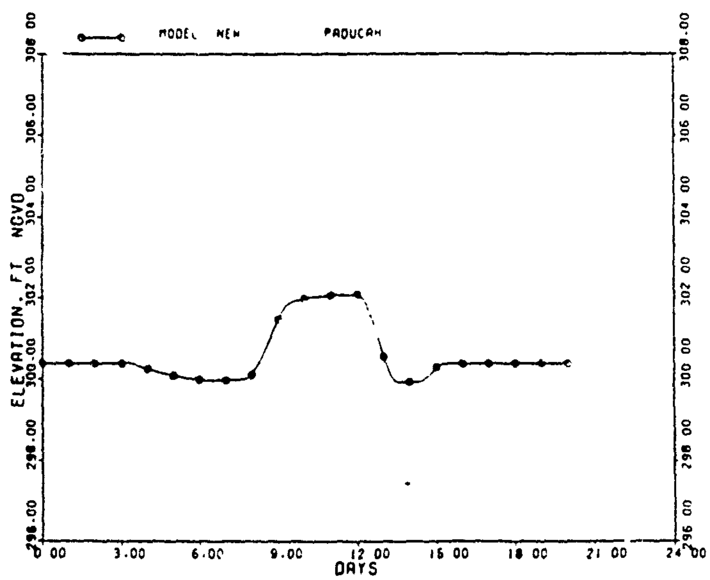


Figure 13. Computed Elevation at Paducah, Kentucky for Smithland Flow Event

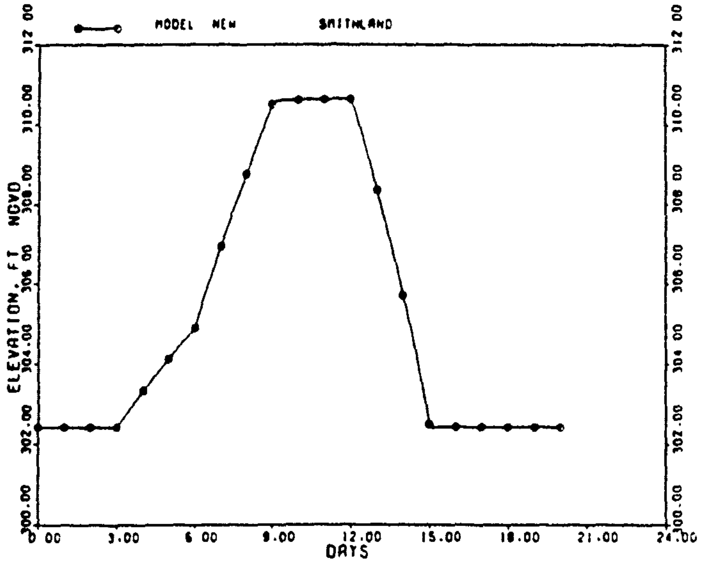


Figure 14. Computed Smithland Tailwater for Smithland Flow Event

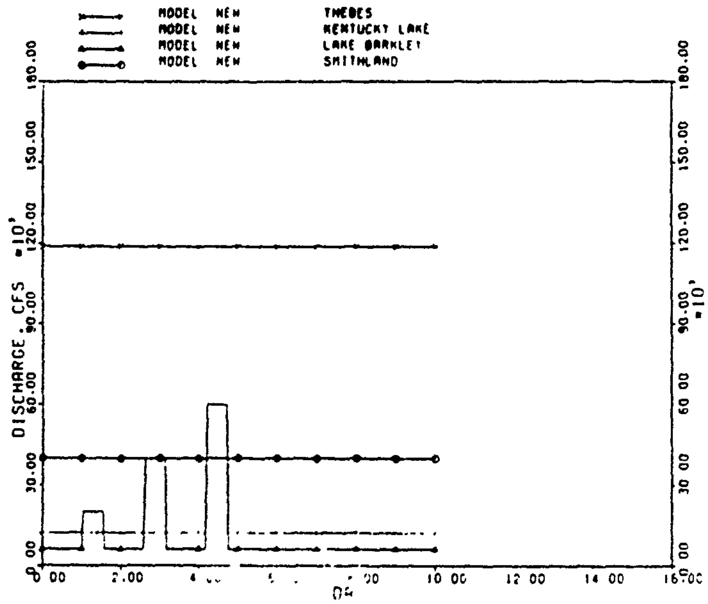


Figure 15. Inflows for Barkley Flow Event

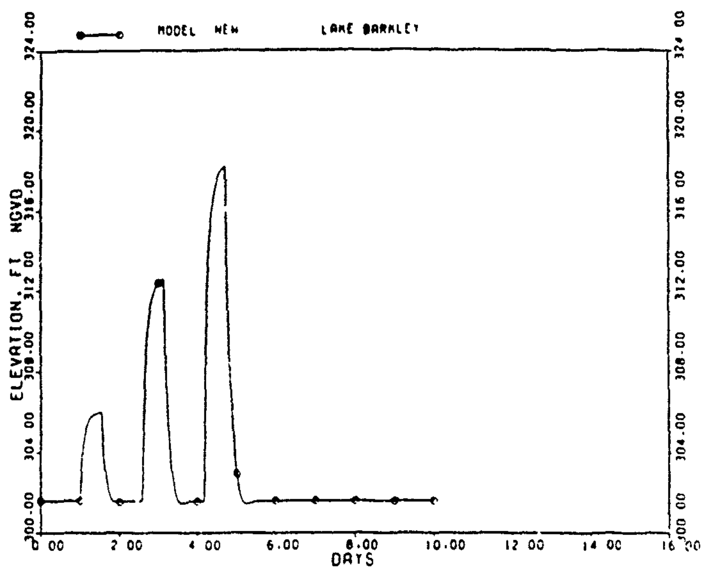


Figure 16. Computed Barkley Tailwater for Barkley Flow Event

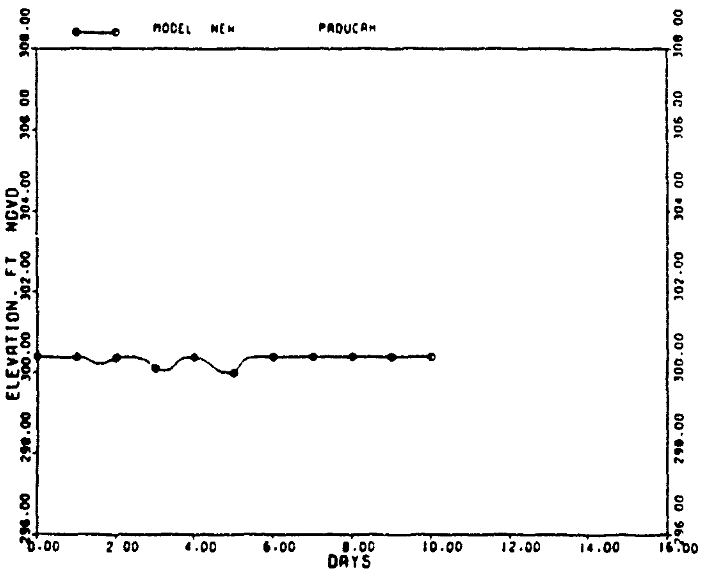


Figure 17. Computed Elevations at Paducah, Kentucky for Barkley Flow Event

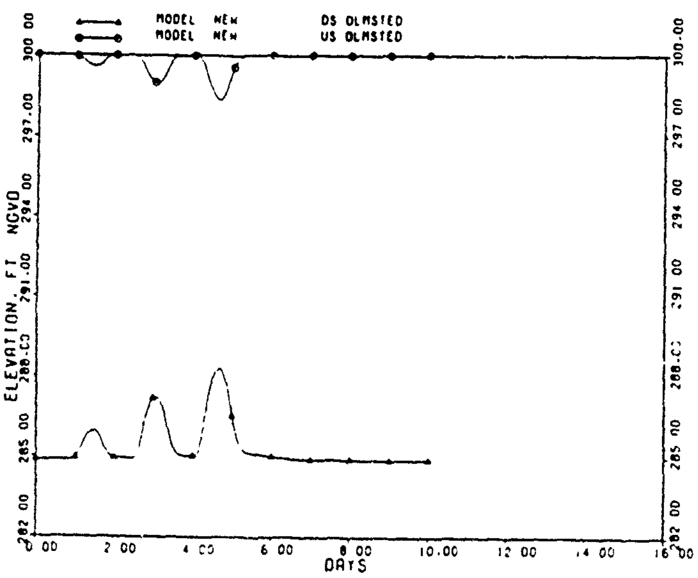


Figure 18. Computed Olmsted Headwater and Tailwater for Barkley Flow Event

pool" with the hinge point being Paducah, Kentucky which is located 30 miles upstream from the damsite. During locking periods the Olmsted headwater elevation will be maintained between elevations 295-300 feet in order to maintain a near constant pool elevation of 300 feet at Paducah, Kentucky. An experimental "hinged pool algorithm" was developed and introduced into the FLOWSED model as a subroutine. Several low flow events were simulated with Locks and Dams 52 and 53 removed and with the proposed Olmsted Project inplace. Based on the results of these simulations the hinged pool algorithm appears to be operating properly and yields reasonable results.

Future Work. Since the hinged pool algorithm was applied to a small number of flow events and those selected events may not cover the full range of reasonably anticipated events in the lower Ohio River, it is believed that some adjustments to the algorithm to reflect actual operating conditions will probably be warranted. Upgrading the reliability of the discharge rating for the Smithland Dam gates should improve the model's performance.

It is anticipated that the modified FLOWSED model will become part of a total project operating system for Olmsted Dam in which most of the gate controls will be automated. For this to be feasible, some refinements to the hinged pool algorithm will be needed to minimize the frequency of gate changes and to provide for smooth operations.

REFERENCES

Chen, Y.H. 1973 (Mar). "Mathematical Modeling of Water and Sediment Routing in Natural Channels," Ph.D. Dissertation at Colorado State University, Fort Collins, CO.

Johnson, B.H. 1982 (Aug). "Development of a Numerical Modeling Capability for the Computation of Unsteady Flow on the Ohio River and Its Major Tributaries," Technical Report HL-82-20, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Johnson, B.H. and Weisinger, L.L. 1990. "Numerical Modeling of Unsteady Flows Through the Proposed Olmsted Hinged Pool," Draft Miscellaneous Paper HL-90- , US Army Engineer Waterways Experiment Station, Vicksburg, MS.

US Army Corps of Engineers 1990 (Apr). "Supplement No. 1 to Design Memorandum No. 1, Main Report, Volume I and Appendices, Olmsted Locks and Dam (Replacement of Locks and Dams 52 and 53)," US Army Corps of Engineers, Louisville District, Louisville, KY.

Flow Regulation Model for the Proposed Hinged Pool Operation, Olmsted Locks and Dam, Ohio River

by

Lyndon C. Richardson, Jr.

SUMMARY OF DISCUSSION BY BRUCE C. BEACH

In response to questions, the author stated the feasibility study considered hydropower, but it was found infeasible. A suitable location in the dam was designed so that hydropower could be added at a later date. Only one other hinged pool exists, at Pittsburgh.

In a discussion, general concern for the operability and reliability of the wicket gates was expressed. The author stated that a similar design exists in France and that prototype wicket gates were to be installed at a facility with similar head to allow for extensive testing.

SESSION II
CONSERVATION STORAGE ANALYSIS

SUMMARY OF SESSION II
CONSERVATION STORAGE ANALYSIS

prepared by

Loren W. Pope
Little Rock District

OVERVIEW

Topics presented in this session consisted of one presentation on the effects of off channel storage on peak flows and the inclusion of this effect into the design of the project, and four presentations dealing primarily with problems associated with conservation storage in multi-purpose reservoirs. The problems included those associated with drought, reallocation and hydropower. Problems such as these will become more prevalent as we place more and more demands on our limited water resources.

PAPER PRESENTATIONS

Olga Boberg, Albuquerque District, presented a paper entitled "Impacts of Gravel Pit Storage at Roswell, New Mexico." Ms. Boberg's paper describes the hydrology and hydraulics of a flood control study for Roswell, NM. She presented the hydrologic model and its calibration in detail and explains how the gravel pits were having a considerable impact on reducing the peak of the 100-year flood. She also describes the existing conditions with some prior channelization and most of the flooding being caused by overflow from the perched Rio Hondo River. The most cost effective alternative was determined to be one that utilizes the capacity of the existing channel through town as well as an existing gravel pit adjacent to the Rio Hondo.

Cecil P. Davis, South Atlantic Division, presented a paper entitled, "Drought Contingency Planning." This paper presents a thorough review of the water management practices relative to drought contingency planning and management. Key issues that surfaced were (1) time required to study and develop DCP's, (2) authority to manage for purposes not specially listed in authorizing legislation, and (3) management for a purpose that was authorized but has no cost allocated to it. Primary finding was that DCP prepared prior to the drought was certainly desirable as it facilitated better public relations.

Ralph R. Hight, Tulsa District, presented a paper entitled, "Reallocation Impacts on Hydropower at Texoma." A reallocation study was conducted to reassign 77,400 acre-feet of power storage in Lake Texoma to satisfy water supply needs. Mr. Hight presented the key issues and impacts of this reallocation. The primary issues presented were the financial settlement with the hydropower interests and whether or not the Secretary of the Army had approval authority. It was determined that the Secretary of the Army had the discretionary approval authority even though the total storage was 150,000 acre-feet. The presentation on the financial settlement was very interesting and enlightening. The final settlement amounted to new thermal replacement value plus an automatic escalation of five percent per year.

D. James Fodrea, North Pacific Division, presented a paper entitled "Determining Dependable Capacity Losses for Water Supply Reallocation Studies." Dependable capacity is necessary in determining the project's contribution to the system's peak load-carrying capacity. It is also needed in determining economic feasibility and in negotiating hydropower sales contracts. Mr. Fodrea presented four methods of determining dependable capacity. He also described where each should be used. The four methods are (1) the critical month method, (2) the firm energy method, (3) the specified availability method, and (4) the average availability method. The last method average availability is recommended for estimating the dependable capacity of hydro plants in large, diverse thermal-based power systems, which are typical of most power systems in the United States.

Werner C. Loehlein, Pittsburgh District, presented a paper entitled "Reallocation of Reservoir Storage for Water Supply Issues and Impacts." In this presentation Mr. Loehlein presented two case studies one on the Allegheny Reservoir and one on Youghiogheny River Lake. A daily flow simulation model was developed and utilized for the studies of Allegheny Reservoir. The studies indicated that due to changes in water demands placed on the basin it appears up to 83,500 acre-feet of storage could be made available for water supply storage. For the studies on the Youghiogheny River a five-day flow simulation model was utilized. From this study it was determined there was no surplus storage for water supply and that the only viable alternative was to increase the summer conservation pool and to make structural modifications to the dam.

IMPACTS OF GRAVEL PIT STORAGE AT ROSWELL, NEW MEXICO

by

OLGA BOBERG[1]

INTRODUCTION: In May of 1988 the Albuquerque District completed work on the Reconnaissance Report for Roswell, New Mexico. The findings of this report were that a significant flood threat existed and that further study was warranted. In the spring of 1989 the feasibility study for Roswell was initiated. The purpose of the study was to develop hydrologic and hydraulic information, determine the magnitude and source of the flooding problems, and develop viable solutions to the problems, potentially in the form of a flood control project for Roswell.

Roswell, New Mexico is subjected to the flows from two rivers which feed the Pecos River, a watershed comprising about 44,000 square miles. In order to define the source of the flooding problems, the hydrology and hydraulics of the individual rivers, as well as their interaction, needed to be understood. A major issue that arose was how to model the gravel pits located in the project area.

The storage effects of the gravel pits proved to have a significant impact on attenuation of peak flows on the North Spring River. This led to the finding that the Rio Hondo was the major contributor to the flooding problems of Roswell. Isolating the source of the flooding problems allowed for the proper development of project alternatives for Roswell.

BASIN DESCRIPTION: Roswell is located in the southeastern part of New Mexico in the Pecos River Watershed. Refer to the vicinity map on Plate 1. The source of the Pecos River basin is in the Sangre de Cristo Mountains about 395 miles north of Roswell. Tributary watersheds in the vicinity of Roswell include Rio Hondo, North Spring River, and Berrendo Creek. Elevations vary from approximately 3443 feet at the confluence of the Rio Hondo with the Pecos River to about 12,000 feet in the upper Rio Hondo watershed.

Rio Hondo is formed at the confluence of the Rio Ruidoso and Rio Bonito, near the village of Hondo in the foothills region of the Sierra Blanca Mountains. Refer to the watershed map on Plate 1. From this point it flows eastward for about 81 miles to its confluence with the Pecos River, 7 miles east of Roswell. The stream is perennial from its source to about the Lincoln-Chaves county line. From this point it is intermittent to the U.S. Army Corps of Engineer's (COE) Two Rivers

[1] Hydraulic Engineer, Albuquerque District, U.S. Army Corps of Engineers

Reservoir and intermittent from the dam to the mouth. The river has been controlled by Two Rivers Dam since 1963 but runoff originating below the dam still causes flooding problems. Refer to the watershed map on Plate 2. The channel capacity of the Rio Hondo still remains very small through Roswell. In most areas, flood damages will occur with any flood larger than about 700 c.f.s. The size of the Rio Hondo drainage area from below the Two Rivers Dam to it's confluence with the North Spring River is 63 square miles.

North Spring River has its source in the low hills about 6 miles west of Roswell. Refer to the watershed map on Plate 3. The drainage system is ill-defined in the upper reaches and consists of a group of broad, shallow draws which converge into a well-defined channel near the western edge of Roswell. From this point the stream continues eastward through the irrigated area west of Roswell to its confluence with the Rio Hondo. North Spring River has a drainage area of 28 square miles.

The Berrendo Creek watershed begins on the eastern slopes of the Capitan Mountains between Hondo and Arabela, New Mexico. From this point it flows eastward for about 56 miles to its confluence with the Rio Hondo about 3 miles east of Roswell. The size of the drainage area is 518 square miles. Berrendo Creek does not contribute to flooding in Roswell.

DISCHARGE FREQUENCY ANALYSIS: There is one gage in the project area: North Spring River at Roswell (the Rio Hondo at Roswell gage period of record is only five years). The period of record for the gage is 1958 to 1986. It is located upstream from Montana Avenue and 2 blocks north of West Second Street in Roswell. Refer to Table 1 for the results of the frequency analysis. The frequency analysis yielded a high standard deviation. Also, it was determined that the discharge record has been significantly affected by the storage effects of the several gravel pits that exist along the river (see Plate 3). Therefore, the results of the frequency analysis of the North Springs gage were not considered reliable.

Twenty-four U.S. Geological Survey streamflow-gaging stations were studied to determine which gages could be utilized in a regional peak frequency analysis. A statistical analysis of the gage records was performed in accordance with Bulletin 17B of the U.S. Water Resources Council, "Guidelines for Determining Flood Flow Frequency". The skew coefficients were obtained based on the report "Generalized Skew Coefficients of Annual Maximum Streamflow Logarithms in Southwestern Division, Corps of Engineers", March 1978. The frequency curves developed were adjusted for expected probability. A multiple linear regression analysis was performed using variables of drainage area size, slope, length of basin, and gage elevation. The 24 gages were reduced to only 7 because of deletions of gages due to elevation, short record length, and drainage area too large. Regression equations were developed for the 100-year and 10-year events. It was determined that the regression equations could not account for the storage effects of the

TABLE 1

NORTH SPRING RIVER AT ROSWELL, NM
DRAINAGE AREA = 19.5 SQ.MI., DATUM OF GAGE IS 3575 FT
GAGE 08393600 - PERIOD OF RECORD 1958 TO 1986
APRIL 1989 CREST GAGE

FINAL RESULTS
-FREQUENCY CURVE-

FLOW, CFS COMPUTED	FLOW, CFS EXPECTED PROBABILITY	EXCEEDANCE PROBABILITY	CONFIDENCE LIMITS .05 LIMIT	CONFIDENCE LIMITS .95 LIMIT
21600.	60700.	.002	210000.	4910.
8840.	18700.	.005	67300.	2330.
4320.	7620.	.010	27100.	1280.
2020.	3050.	.020	10400.	673.
891.	1180.	.040	3700.	335.
265.	308.	.100	826.	116.
90.	97.	.200	226.	44.
13.	13.	.500	27.	7.
2.	2.	.800	5.	1.
1.	1.	.900	2.	0.
1.	0.	.950	1.	0.
0.	0.	.990	1.	0.

FREQUENCY CURVE STATISTICS		STATISTICS BASED ON	
MEAN LOGARITHM	1.1894	HISTORIC EVENTS	1
STANDARD DEVIATION	.9380	HIGH OUTLIERS	0
COMPUTED SKEW	.5694	LOW OUTLIERS	0
GENERALIZED SKEW	.1000	ZERO OR MISSING	7
ADOPTED SKEW	.3896	SYSTEMATIC EVENTS	27
		HISTORIC PERIOD	33

gravel pits located in the study area. This made it necessary to develop a rainfall runoff model using the U.S. Army Corps of Engineers HEC-1 flood hydrograph package for determining frequency flow data for Roswell.

UPPER BASIN MODEL CALIBRATION: From the frequency analysis, three gages located in the upper basin of the Rio Hondo were selected as being reliable for use in calibrating a model of the upper basin above Roswell. Refer to Tables 2-4 for the results of the frequency analysis. An HEC-1 model was developed of the Rio Ruidoso, Rio Bonito, and Rio Hondo watersheds with concentration points at the gages: Rio Bonito at Hondo, Rio Ruidoso at Hondo, and Rio Hondo at Diamond A Ranch. After the Snyder unit hydrograph parameters were selected, then watershed loss rates were determined by calibrating the model to the discharge frequency relationships that had been previously developed for the three gaged watersheds. The peaks in the calibration model were matched to the peaks in the frequency analysis with emphasis placed on matching to the Rio Hondo at Diamond A Ranch gage(a basin area of 947 square miles). Results of the calibration are shown on Table 5. Refer to the watershed map on Plate 2 for locations of gages. A field survey provided information with which to design the hydrologic model. Refer to Plates 4-7 for frequency curves of the gages. The initial losses obtained from this calibration were utilized in the Roswell HEC-1 model.

UNIT HYDROGRAPHS: The Snyder synthetic unit hydrograph method was utilized. The relationship between Ct and slope developed for previous studies of watersheds in New Mexico and Texas by the Albuquerque District was adopted for use in the Roswell study. Flood reconstitution data of streams in the Pecos River Basin were used to verify the Ct curves's applicability to the study area. A Cp value of .8 was chosen for the model also based on flood reconstitution data. Plate 8 shows Snyder's Ct versus equivalent slope curve.

INFILTRATION RATES: A constant loss rate of .25 inch/hour was used for the calibration model based on an approximate study of the soil types in the basin using Soil Conservation Service (SCS) Soil Surveys of Chaves and Lincoln County, New Mexico as well as New Mexico State University's Research Report: Soils of New Mexico. The drainage basin was found to contain primarily SCS type B and C soils corresponding to SCS infiltration indices in the .05-.3 inch/hour range. Flood reconstitution data of streams in the Pecos River watershed indicates that constant loss rates in the range of .2-.4 inches/hour are possible. The calibration model yielded initial loss rates for the 10-year, 50-year, and 100-year of 1.10 inch, 1.00 inch, and .60 inch respectively. These values were applied to the Roswell model using a constant loss rate of .25 inch/hour. An initial loss rate of zero inches and a constant loss rate of .25 inches/hour were used for the Standard Project Flood model. Percent of impervious area for a 100-year future growth projection was estimated using a 50-year future growth projection developed by the City Planner of the city of Roswell.

TABLE 2

RIO BONITO @ HONDO, NM
DRAINAGE AREA = 295.0 SQ.MI., DATUM OF GAGE IS 5205 FT
GAGE 08389500 - PERIOD OF RECORD 1931 TO 1967
APRIL 1989

FINAL RESULTS
-FREQUENCY CURVE-

FLOW, CFS COMPUTED	FLOW, CFS EXPECTED PROBABILITY	EXCEEDANCE PROBABILITY	CONFIDENCE LIMITS .05 LIMIT	CONFIDENCE LIMITS .95 LIMIT
46500.	58700.	.002	107000.	25900.
34500.	41200.	.005	74100.	20000.
26800.	30900.	.010	54600.	16100.
20200.	22600.	.020	38900.	12600.
14700.	16000.	.040	26600.	9530.
8920.	9330.	.100	14700.	6090.
5500.	5630.	.200	8360.	3910.
2100.	2100.	.500	2910.	1520.
766.	746.	.800	1080.	505.
443.	421.	.900	552.	268.
279.	257.	.950	432.	155.
115.	95.	.990	199.	53.

FREQUENCY CURVE STATISTICS		STATISTICS BASED ON	
MEAN LOGARITHM	3.3081	HISTORIC EVENTS	0
STANDARD DEVIATION	.5092	HIGH OUTLIERS	0
COMPUTED SKEW	-.2529	LOW OUTLIERS	0
GENERALIZED SKEW	-.0054	ZERO OR MISSING	0
ADOPTED SKEW	-.1726	SYSTEMATIC EVENTS	37

TABLE 3

RIO RUIDOSO @ HONDO, NM
DRAINAGE AREA - 290.0 SQ.MI., DATUM OF GAGE IS 5181 FT
GAGE 08388000 - PERIOD OF RECORD 1931 TO 1970
APRIL 1989

FINAL RESULTS
-FREQUENCY CURVE-

FLOW, CFS COMPUTED	FLOW, CFS EXPECTED PROBABILITY	EXCEEDANCE PROBABILITY	CONFIDENCE LIMITS .05 LIMIT	CONFIDENCE LIMITS .95 LIMIT
48100.	65700.	.002	126000.	24200.
31400.	39600.	.005	75500.	16700.
22100.	26500.	.010	49500.	12400.
15100.	17300.	.020	31400.	8890.
9970.	11000.	.040	19100.	6150.
5260.	5540.	.100	8970.	3480.
2910.	2990.	.200	4530.	2020.
958.	958.	.500	1350.	678.
325.	317.	.800	468.	208.
186.	178.	.900	281.	110.
119.	110.	.950	187.	65.
51.	44.	.990	90.	24.

FREQUENCY CURVE STATISTICS		STATISTICS BASED ON	
MEAN LOGARITHM	2.9901	HISTORIC EVENTS	0
STANDARD DEVIATION	.5661	HIGH OUTLIERS	0
COMPUTED SKEW	.1466	LOW OUTLIERS	0
GENERALIZED SKEW	-.0396	ZERO OR MISSING	0
ADOPTED SKEW	.0912	SYSTEMATIC EVENTS	40

TABLE 4

RIO HONDO @ DIAMOND A RANCH NEAR ROSWELL, NM
DRAINAGE AREA = 947.0 SQ.MI., DATUM OF GAGE IS 4190 FT
GAGE 08390500 - PERIOD OF RECORD MAY 1939 TO 1987
APRIL 1989

FINAL RESULTS
-FREQUENCY CURVE-

FLOW, CFS COMPUTED	FLOW, CFS EXPECTED PROBABILITY	EXCEEDANCE PROBABILITY	CONFIDENCE LIMITS .05 LIMIT	CONFIDENCE LIMITS .95 LIMIT
236000.	318000.	.002	598000.	120000.
143000.	179000.	.005	330000.	77300.
95500.	113000.	.010	204000.	54200.
61900.	70100.	.020	123000.	37000.
38600.	42100.	.040	70500.	24400.
19000.	19900.	.100	31000.	12900.
10000.	10200.	.200	15000.	7110.
3120.	3120.	.500	4290.	2260.
1060.	1030.	.800	1490.	701.
618.	595.	.900	905.	382.
403.	381.	.950	615.	234.
188.	168.	.990	312.	96.

FREQUENCY CURVE STATISTICS		STATISTICS BASED ON	
MEAN LOGARITHM	3.5188	HISTORIC EVENTS	0
STANDARD DEVIATION	.5819	HIGH OUTLIERS	0
COMPUTED SKEW	.3766	LOW OUTLIERS	0
GENERALIZED SKEW	-.0492	ZERO OR MISSING	0
ADOPTED SKEW	.2539	SYSTEMATIC EVENTS	49

TABLE 5

HEC-1 MODEL FREQUENCY FLOWS
CALIBRATED TO USGS STREAM GAGES

LOCATION	DRAINAGE AREA (Sq.MI.)	10-YEAR GAGE (CFS)	10-YEAR HEC-1 (CFS)	50-YEAR GAGE (CFS)	50-YEAR HEC-1 (CFS)	100-YEAR GAGE (CFS)	100-YEAR HEC-1 (CFS)
Rio Bonito @ Hondo	295	9300	5900	22600	18500	30900	26100
Rio Ruidoso @ Hondo	290	5500	6600	17300	20500	26500	29000
Rio Hondo @ Diamond A	947	19900	19600	70100	69200	113000	108600

ROSWELL MODEL CALIBRATION: In addition to the upper basin calibration of the model to the frequency curves mentioned above, the model was used to reproduce the 1954 peak on North Spring River. During May 17 and 18 of 1954, Roswell experienced a large storm event over North Spring River. The resulting flood was due to a thunderstorm that concentrated its heaviest precipitation directly over the North Spring River drainage basin and part of the Berrendo Creek basin.

According to the flood report prepared by the Albuquerque District, the storm period began at 6:50 P.M. on May 17 and continued until 7:45 A.M. on May 18. Precipitation during the storm was recorded by the National Weather Service at the Roswell Municipal Airport. An isohyetal map was developed from rainfall information obtained by individuals during the storm. Table 6 shows rainfall data of the 1954 Storm. A peak discharge of 7,000 c.f.s. was estimated at Wyoming Street in Roswell using the slope-area method. As a check, the 1954 storm rainfall distribution and rainfall amounts were applied to the North Spring River HEC-1 model and a peak discharge of 7,500 c.f.s. was computed at the concentration point near Wyoming Street. Plate 9 shows the HEC-1 model hydrograph and hyetograph at Wyoming Street.

RAINFALL: Frequency flow data for Roswell was computed by application of frequency rainfall to the calibrated HEC-1 Roswell model. Point precipitation rainfall values for the 10-year, 50-year, and 100-year events were obtained from the NOAA Atlas 2, Volume IV-New Mexico.

TABLE 6
STORM OF 17-18 MAY 1954

PRECIPITATION RECORDED AT ROSWELL MUNICIPAL AIRPORT

DATE	TIME	PRECIPITATION (Inches)	DATE	TIME	PRECIPITATION (Inches)
17 May	7:00 p.m.	0.08	18 May	12:00 a.m.	Trace
"	8:28 p.m.	0.05	"	1:00 a.m.	0.02
"	9:00 p.m.	1.08	"	2:00 a.m.	Trace
"	10:00 p.m.	1.10	"	3:00 a.m.	Trace
"	11:28 p.m.	0.16	"	4:00 a.m.	0.01
			"	5:28 a.m.	Trace
			"	6:28 a.m.	Trace
			"	7:28 a.m.	Trace

RAINFALL DISTRIBUTION PATTERN

DATE	TIME	TOTAL RAINFALL (Inches)	RAINFALL (Inches)	DATE	TIME	TOTAL RAINFALL (Inches)	RAINFALL (Inches)
5/17	6:45 pm	0	0	5/18	12:00 am	2.47	0
"	7:00 pm	0.08	0.08	"	12:15 am	2.47	0
"	7:15 pm	0.08	0	"	12:30 am	2.48	0.01
"	7:30 pm	0.09	0.01	"	12:45 am	2.48	0
"	7:45 pm	0.10	0.01	"	1:00 am	2.49	0.01
"	8:00 pm	0.11	0.01	"	1:15 am	2.49	0
"	8:15 pm	0.12	0.01	"	1:30 am	2.49	0
"	8:30 pm	0.13	0.01	"	1:45 am	2.49	0
"	8:45 pm	0.70	0.57	"	2:00 am	2.49	0
"	9:00 pm	1.21	0.51	"	2:15 am	2.49	0
"	9:15 pm	1.55	0.34	"	2:30 am	2.49	0
"	9:30 pm	1.85	0.30	"	2:45 am	2.49	0
"	9:45 pm	2.10	0.25	"	3:00 am	2.49	0
"	10:00 pm	2.31	0.21	"	3:15 am	2.49	0
"	10:15 pm	2.35	0.04	"	3:30 am	2.49	0
"	10:30 pm	2.39	0.04	"	3:45 am	2.49	0
"	10:45 pm	2.42	0.03	"	4:00 am	2.50	0.01
"	11:00 pm	2.44	0.02	"	4:15 am	2.50	0
"	11:15 pm	2.46	0.02	"	4:30 am	2.50	0
"	11:30 pm	2.47	0.01	"	4:45 am	2.50	0
"	11:45 pm	2.47	0	"	5:00 am	2.50	0
				"	5:15 am	2.50	0
				"	5:30 am	2.50	0
				"	5:45 am	2.50	0
				"	6:00 am	2.50	0
				"	6:15 am	2.50	0
				"	6:30 am	2.50	0
				"	6:45 am	2.50	0
				"	7:00 am	2.50	0

Uniform rainfall was applied with areal storm size adjustments made according to the procedures described in (National Weather Service, 1973), "National Oceanic and Atmospheric Administration, Precipitation-Frequency Atlas of the Western United States, Volume IV-New Mexico". Frequency flows were obtained using incremental rainfall amounts distributed in a realistic pattern which was verified by comparisons to the mass rainfall curves of storms in the Pecos River watershed. The realistic pattern was developed for the Denver, Colorado area and distributes rainfall amounts such that the maximum increment is placed at the beginning of the second half hour of the storm and the remaining increments are placed so that they ascend in magnitude to the peak and then descend in magnitude to the end. Rainfall data is shown on Table 7. A 24-hour rainfall duration was applied because the Berrendo Creek subarea in the model has a time to peak of almost 8 hours and it was felt that a storm duration longer than this was needed. Also, because of the reservoir routing used for the gravel pit modeling, a storm duration of 24-hours is appropriate. Refer to Table 8 for tabulation of peak flows. Discharge-frequency curves are shown on Plates 10, 11, and 12.

TABLE 7

24-HOUR RAINFALL AMOUNTS

AREA	10-YEAR (Inches)	50-YEAR (Inches)	100-YEAR (Inches)
North Spring River	3.09	4.36	4.90
South Berrendo	2.82	3.93	4.33
Rio Hondo	2.92	4.11	4.57
Upper Basin*	2.66	3.64	4.00

* Upper basin calibrated to USGS stream gages:
Rio Ruidoso @ Hondo
Rio Bonito @ Hondo
Rio Hondo @ Diamond A

STANDARD PROJECT FLOOD: The SPS rainfall was taken to be .4 of the Probable Maximum Precipitation (PMP) rainfall based on the magnitude of storm events experienced in the region. This is in accordance with the range of 40% to 60% specified in EM 1110-2-1411. The PMP amounts were taken from (National Oceanic and Atmospheric Administration, Corps of Engineers, Bureau of Reclamation, June 1988) "Hydrometeorological Report No. 55A, Probable Maximum Precipitation Estimates-United States Between the Continental Divide and the 103rd Meridian." The incremental rainfall amounts for the SPS model were critically ordered with the maximum six-hour SPS distributed critically. A storm was centered over the 610 square mile area of Berrendo Creek, North Spring River, and Rio Hondo (below Two Rivers Dam) watersheds using the SPS isohyetal pattern from the SWD Watershed Runoff Model to obtain SPS rainfall amounts for Berrendo Creek. SPS rainfall amounts for North Spring River and Rio

Hondo (below the dam) were determined using procedures in HMR 55A for application of uniform rainfall. Refer to Table 8 for tabulation of peak flows. Discharge curves on Plates 10, 11, and 12 designate the computed SPF values.

FLOOD ROUTING: The Modified Puls routing method was selected for channel routing of flows. It was determined that Muskingum routing could not be used because there are no hydrographs or other appropriate hydrologic data with which to calibrate a Muskingum X coefficient. Routing of flow through the gravel pits located in the North Spring River and Rio Hondo basins were also modeled using Modified Puls reservoir routing. Estimates of capacities were obtained from 1"=400' mapping with 5' contour intervals and 7.5 minute U. S. Geological Survey quadrangle maps with 10' contour intervals. Table 9 has estimated gravel pit capacities for the gravel pits in the Rio Hondo and North Spring River watersheds. Outflow estimates were made using the standard discharge equation: Q=CLH 3/2 with a discharge coefficient C of 3.00. Weir lengths were estimated from the available mapping.

TABLE 8

FREQUENCY FLOW DATA

LOCATION	DRAINAGE AREA (Sq.Mi.)	10-YEAR (CFS)	25-YEAR (CFS)	50-YEAR (CFS)	100-YEAR (CFS)	SPF (CFS)
North Spring River @ Gage	18.7	700	1,500	2,400	3,900	13,000
North Spring River @ Rio Hondo	28.6	2,300	3,400	4,600	6,600	21,000
Rio Hondo above Roswell	43.2	1,100	3,100	5,800	10,000	33,000
Berrendo Creek @ Rio Hondo	518.0	16,000	29,000	42,000	63,000	134,000

STUDY RESULTS: The storage effects of the gravel pits on the attenuation of the peak discharges of the North Spring River are significant. The 100-year discharge on the North Spring River at it's confluence with the Rio Hondo is 6600 c.f.s. When the gravel pits are removed from the model, the discharge increases to approximately 14,000 c.f.s. The HEC-1 model was used to reproduce the 1954 storm on North Spring River. The results were good. The peak flow at Wyoming Street of 7,500 c.f.s. predicted by the model, was close to the actual peak flow of 7,000 c.f.s. The reproduction of the historical storm was very important in the

verification of the hydrologic modeling. It showed that the model was reasonably accurate in predicting the effects of the gravel pits. The existence of this storm data was so significant because there was no outflow hydrograph data of the gravel pits to use for calibration. The hydrology data developed for the North Spring River had a big impact on the design of flood control project alternatives for Roswell. Channel improvements on the North Spring River made by the City of Roswell and Chaves County Flood Control Association have helped to alleviate flooding caused by flows originating from the North Spring River. Essential to the design of a flood control project for Roswell was knowing what level of protection the existing flood control structure on North Spring River provided. Based on the flows obtained from the HEC-1 model, the amount of flooding resulting from the North Spring River flows alone was not enough to justify the expense of a flood control project. The selected alternative does not include a plan of improvement for North Spring River.

TABLE 9

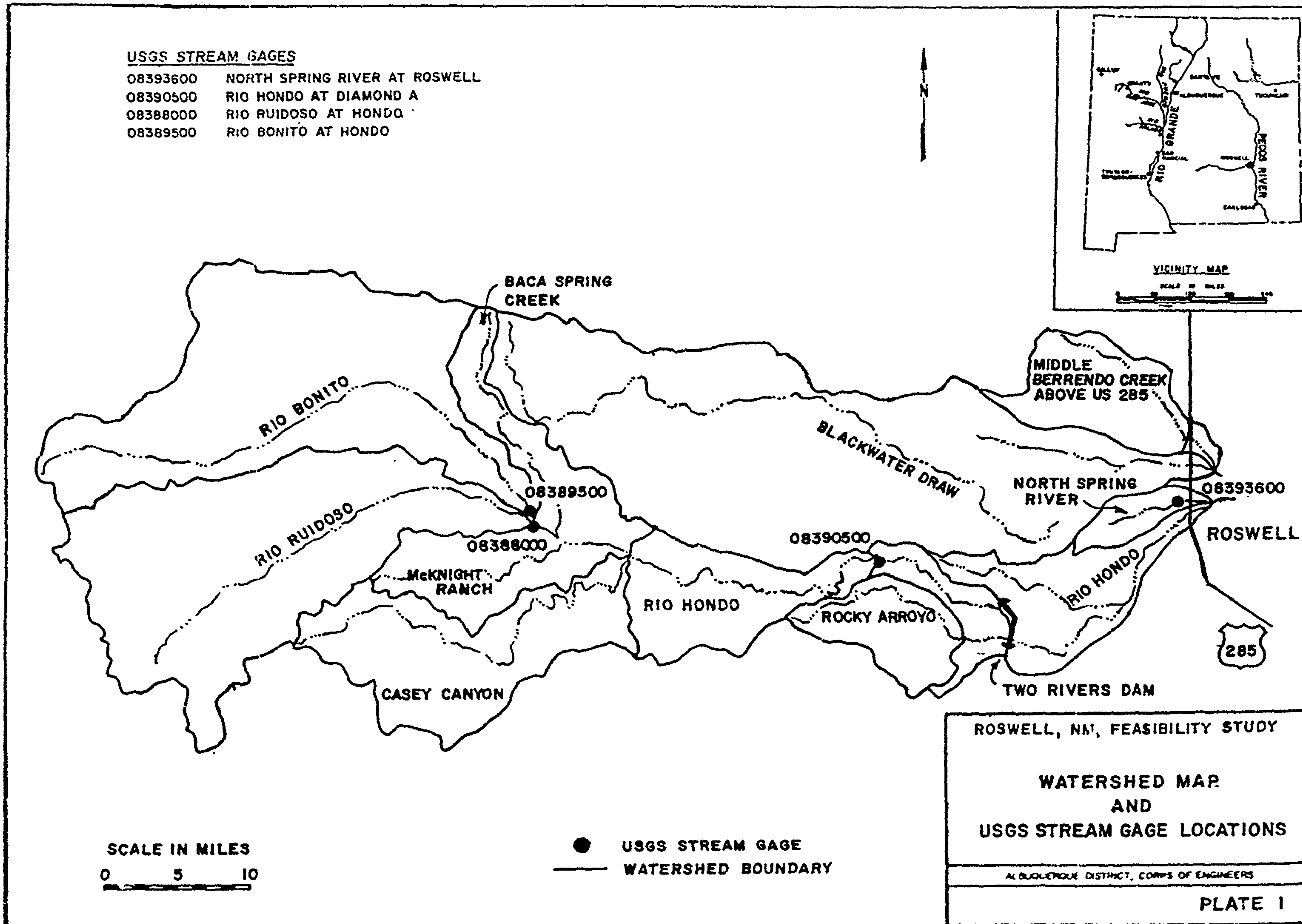
APPROXIMATE GRAVEL PIT CAPACITIES

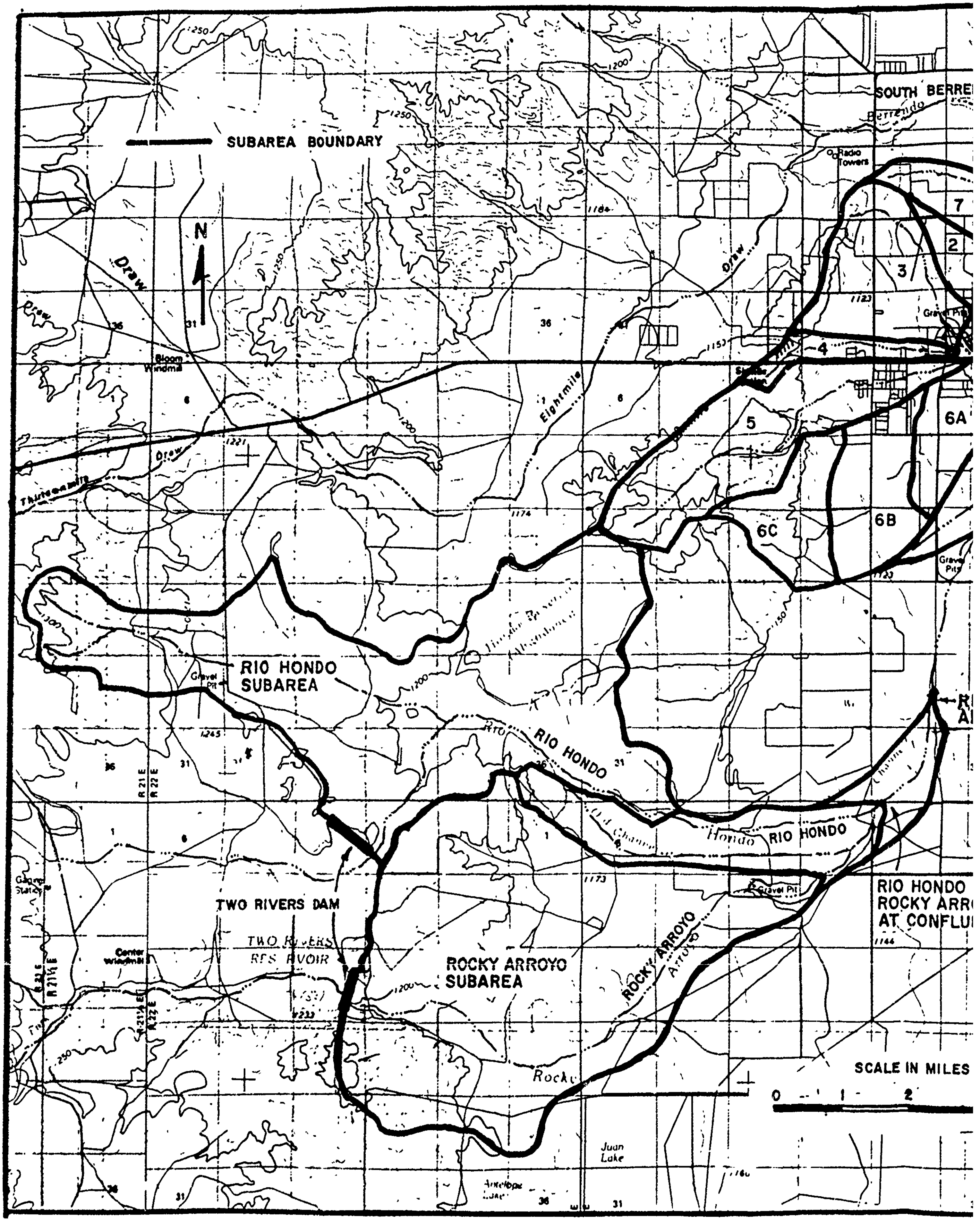
SUBAREA	DRAINAGE AREA (SQ. MI.)	CAPACITY (AC-FT)	100-YEAR RUNOFF VOLUME (AC-FT)
NORTH SPRINGS WATERSHED:			
4	1.1	175	166
3	3.4	300	503
2	1.6	150	237
RIO HONDO WATERSHED:			
ROCKY ARROYO	19.2	454	2069

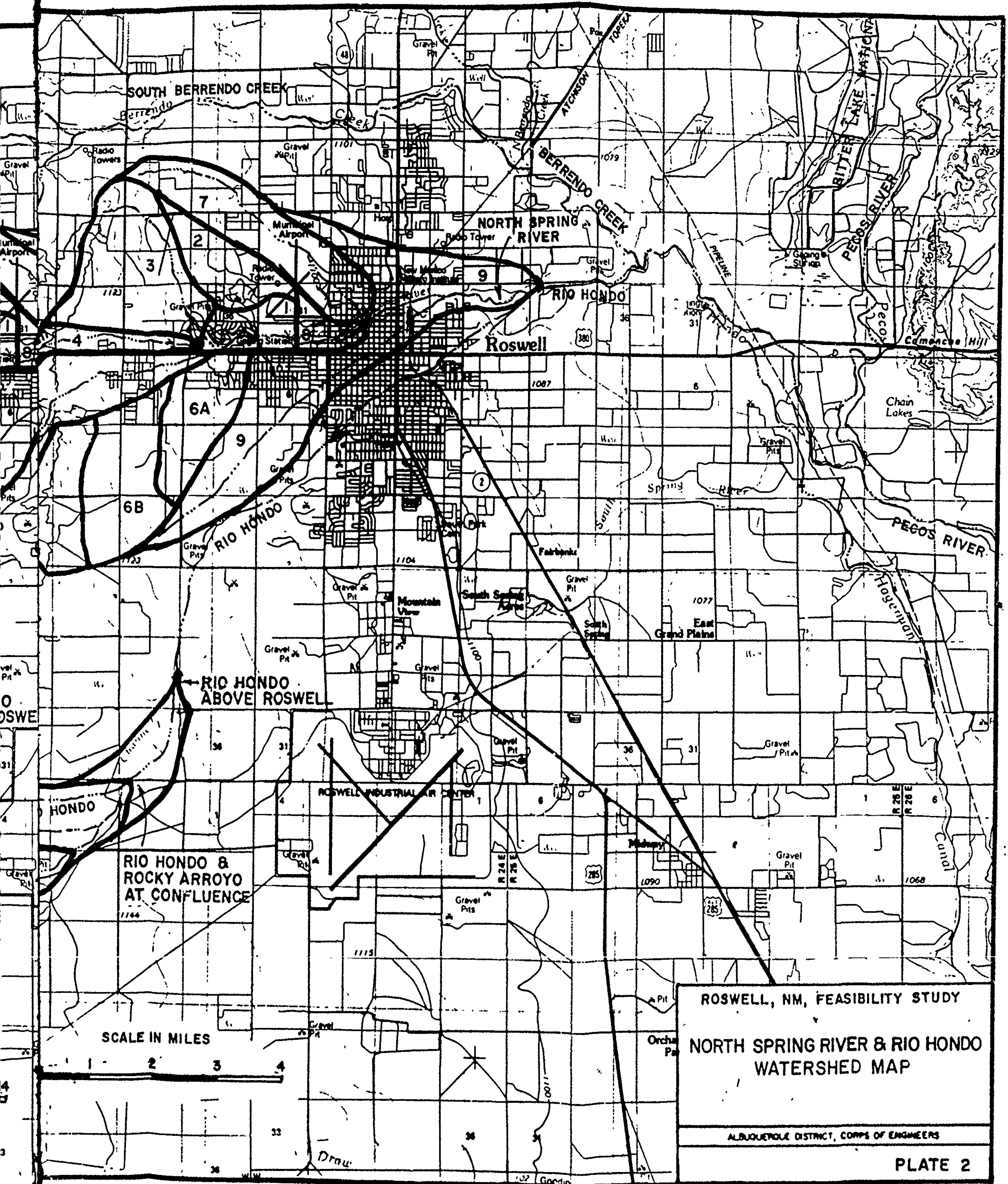
CONCLUSIONS: The major contributor to flooding in Roswell is the Rio Hondo. Throughout the reach of the Rio Hondo through town, the river is perched. The difference in elevation between the inverts of the Rio Hondo and the North Spring River is as much as 15 feet. Flows in excess of the capacity of the Rio Hondo leave the channel flowing northward towards the North Spring River. The flow contribution from the Rio Hondo to the North Spring River is significant. The result is a wide floodplain which is continuous between the two rivers. Most of the flooding along the North Spring River which would occur during a large event, would be due to flows originating from the Rio Hondo. It was felt

that if high flows on the Rio Hondo could be controlled through town, then flooding problems from Hondo flows could be eliminated and the residual flooding from North Spring River flows would be minor. Several alternatives were considered. The most cost effective concept was determined to be the alternative which utilizes the capacity of the existing channel through town as well as an off channel storage utilizing an existing gravel pit site adjacent to the Rio Hondo. The gravel pit area can provide some of the storage required for the detention basin. Since the effects of the gravel pits are significant in terms of attenuation of the peak runoff on North Springs River, the gravel pits need to be considered as an integral part of a project that is implemented. Consideration of the protection provided by the gravel pits needs to be included in the Local Cost Sharing Agreement.

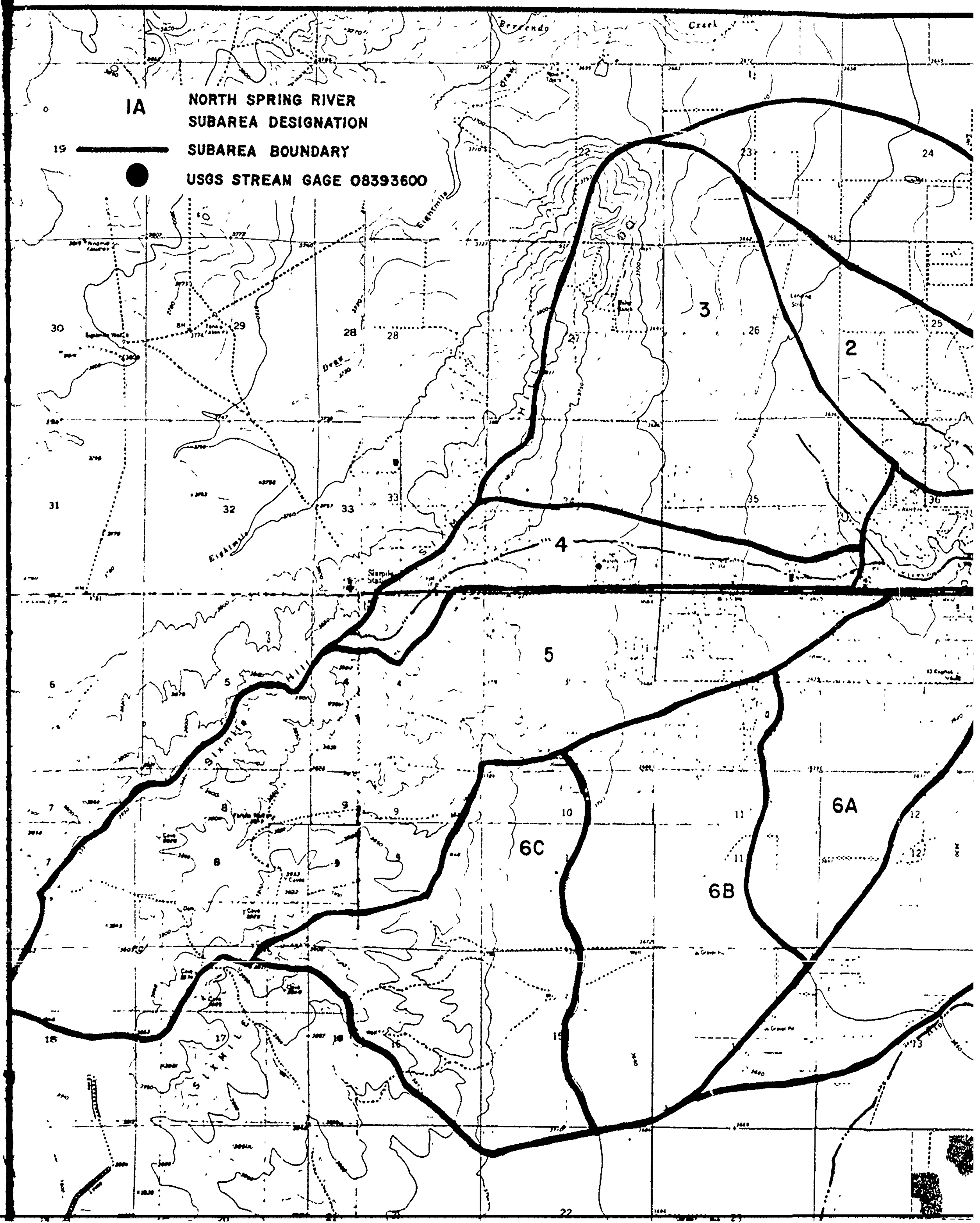
<u>SELECTED ALTERNATIVE:</u> The purpose of the off channel storage alternative is to reduce peak flood discharges to acceptable channel capacity within the urban area. The alternative consists of training levees located at the beginning of an improved earth channel on the Rio Hondo which will convey flows to a detention structure located adjacent to the Rio Hondo. Flow diversions into the storage area will be made by means of a weir structure which will introduce flows in excess of 600 c.f.s. into the detention structure. The head driving the weir will be produced by the backwater effects of a control section to be placed downstream of the weir. The existing Rio Hondo will convey flows of 600 c.f.s. around the detention basin and through town. Releases from storage will be made through a gated outlet works.


USGS STREAM GAGES
08393600 NORTH SPRING RIVER AT ROSWELL
08390500 RIO HONDO AT DIAMOND A
08388000 RIO RUIDOSO AT HONDO
08389500 RIO BONITO AT HONDO
N
PECOS RIVER
RIO GRANDE
VICINITY MAP
BACA SPRING CREEK
MIDDLE BERRENDO CREEK ABOVE US 285
RIO BONITO
BLACKWATER DRAW
NORTH SPRING RIVER
08393600
ROSWELL
08389500
RIO RUIDOSO
08388000
08390500
McKNIGHT RANCH
RIO HONDO
ROCKY ARROYO
RIO HONDO
285
CASEY CANYON
TWO RIVERS DAM
ROSWELL, NM, FEASIBILITY STUDY
WATERSHED MAP
AND
USGS STREAM GAGE LOCATIONS
ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS
PLATE I
SCALE IN MILES
0 5 10
USGS STREAM GAGE
WATERSHED BOUNDARY

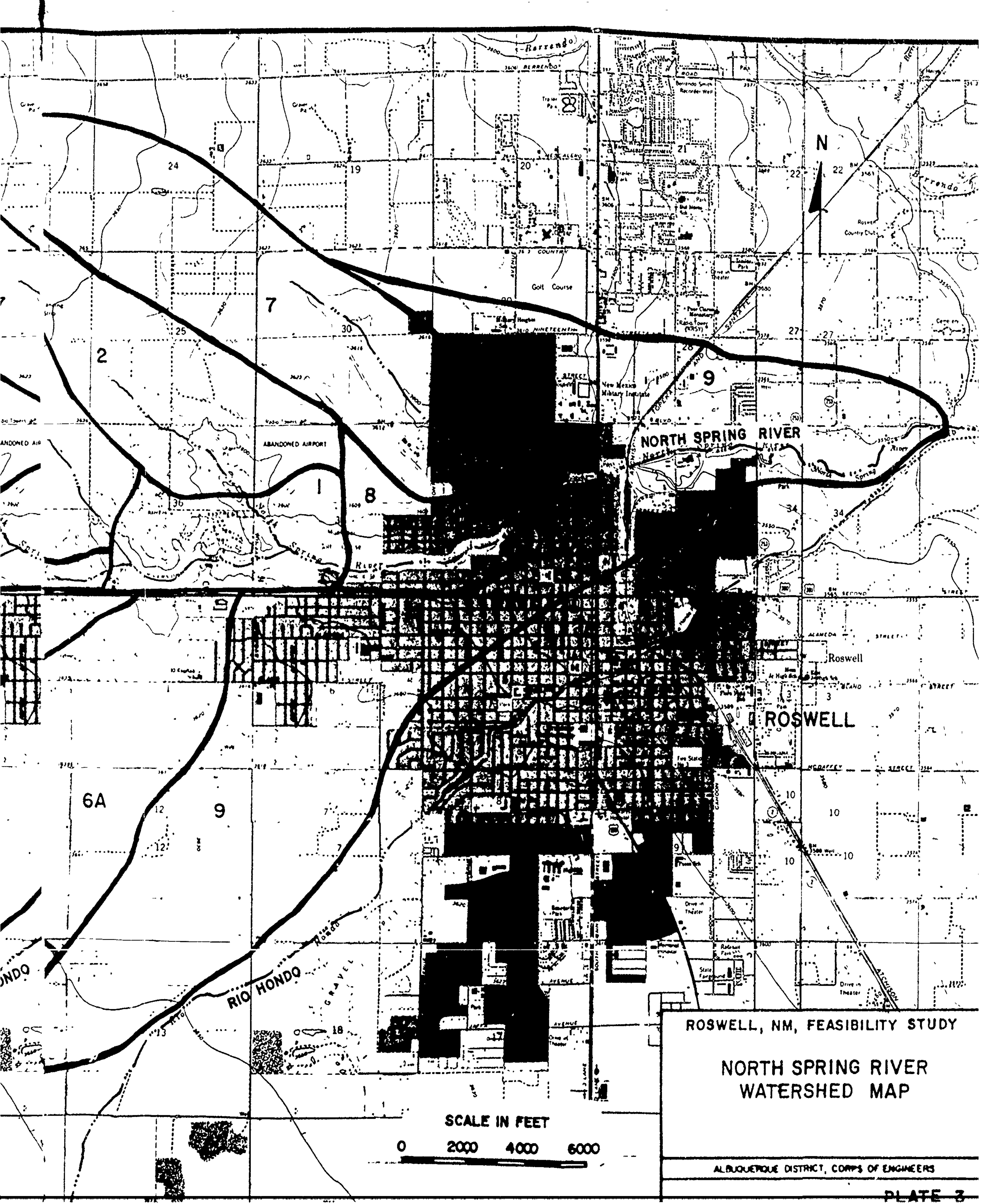

SUBAREA BOUNDARY
N
SOUTH BERRE
Berrendo
Radio Towers
7
2
3
4
5
6A
6B
6C
Gravel Pit
Gravel Pits
Bloom Windmill
Draw
Eightmile
Thirteenmile
Draw
RIO HONDO SUBAREA
Gravel Pit
Rio
RIO HONDO
Hondo
RIO HONDO
Old Channel
TWO RIVERS DAM
TWO RIVERS RESERVOIR
ROCKY ARROYO SUBAREA
ROCKY ARROYO
Arroyo
Rocky
Gravel Pit
RIO HONDO
ROCKY ARRO
AT CONFLU
Center Windmill
Juan Lake
Antelope Lake
SCALE IN MILES
0
1
2
R 21 E
R 22 E



SOUTH BERRENDO CREEK
BERRENDO CREEK
NORTH SPRING RIVER
RIO HONDO
Roswell
PECOS RIVER
BITTER LAKE NATIONAL
Chain Lakes
RIO HONDO ABOVE ROSWELL
RIO HONDO & ROCKY ARROYO AT CONFLUENCE
ROSWELL INDUSTRIAL AIR CENTER
SCALE IN MILES
0 1 2 3 4
ROSWELL, NM, FEASIBILITY STUDY
NORTH SPRING RIVER & RIO HONDO WATERSHED MAP
ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS
PLATE 2



1A NORTH SPRING RIVER SUBAREA DESIGNATION
19 SUBAREA BOUNDARY
USGS STREAN GAGE 08393600
Berrendo
Creek
Eightmile
Draw
Sixmile
Hill
1A
2
3
4
5
6A
6B
6C

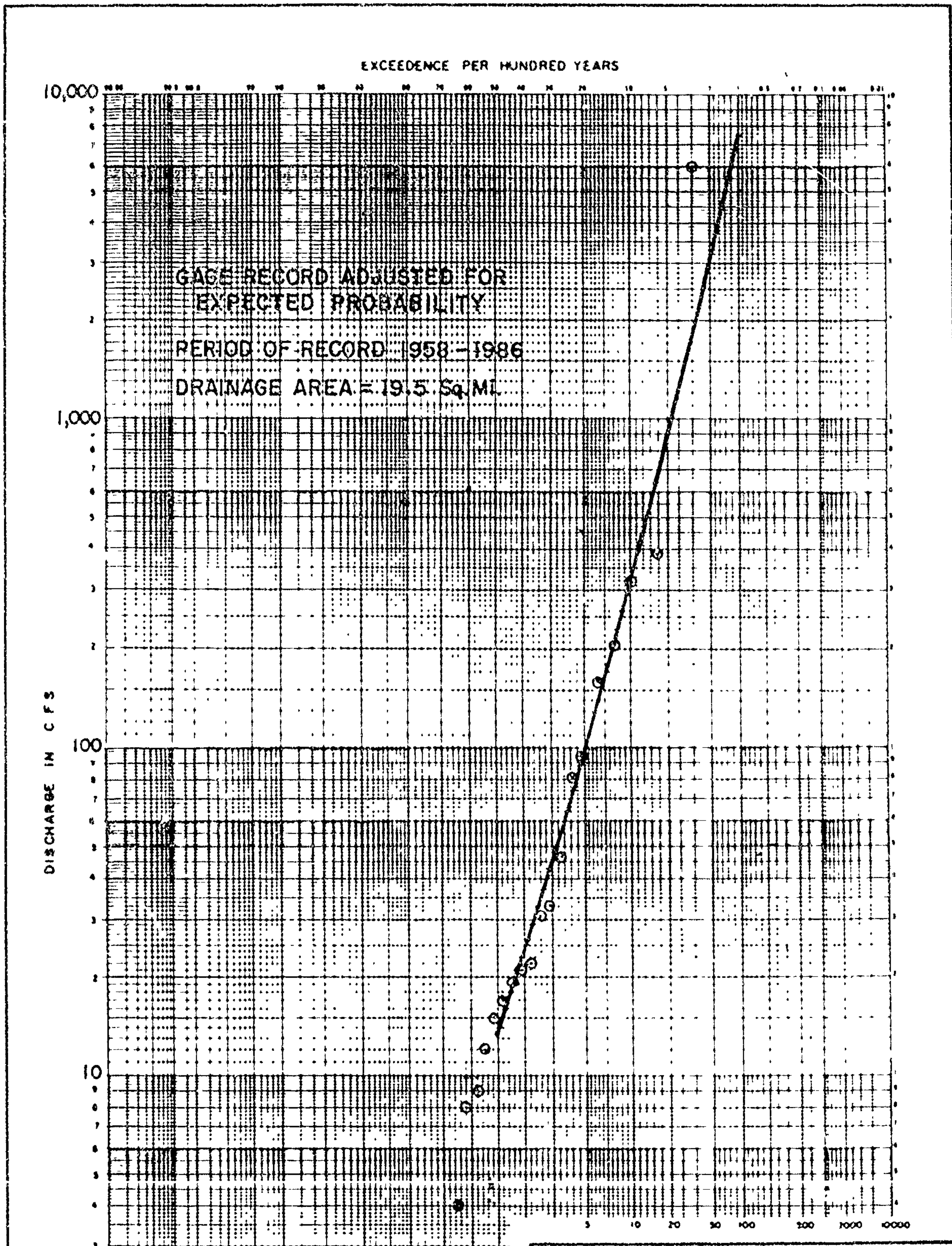


ROSWELL, NM, FEASIBILITY STUDY

NORTH SPRING RIVER
WATERSHED MAP

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 3

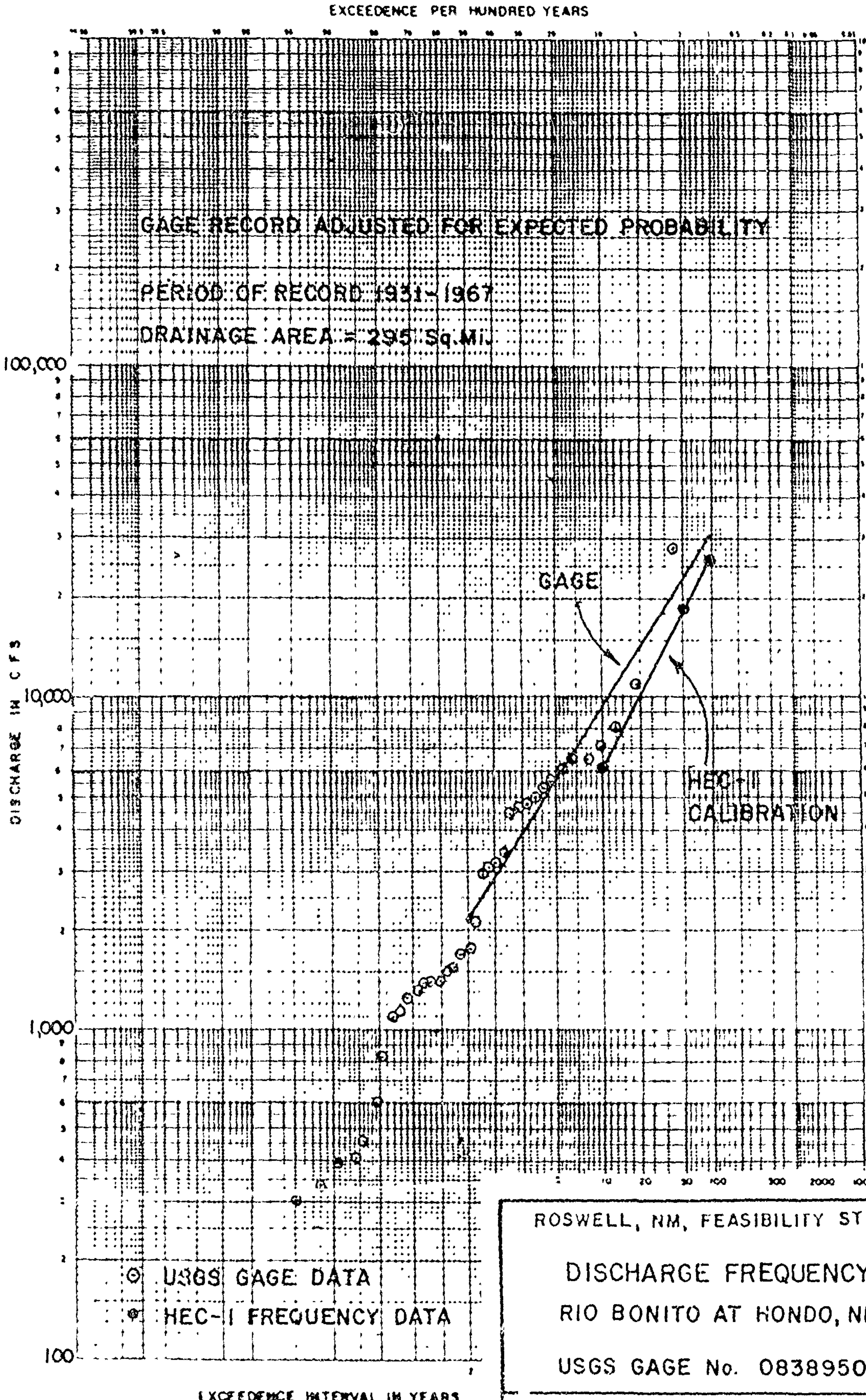


ROSWELL, NM, FEASIBILITY STUDY

DISCHARGE FREQUENCY
NORTH SPRING RIVER
AT ROSWELL, NM
USGS GAGE No. 08393600

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 4



ROSWELL, NM, FEASIBILITY STUDY

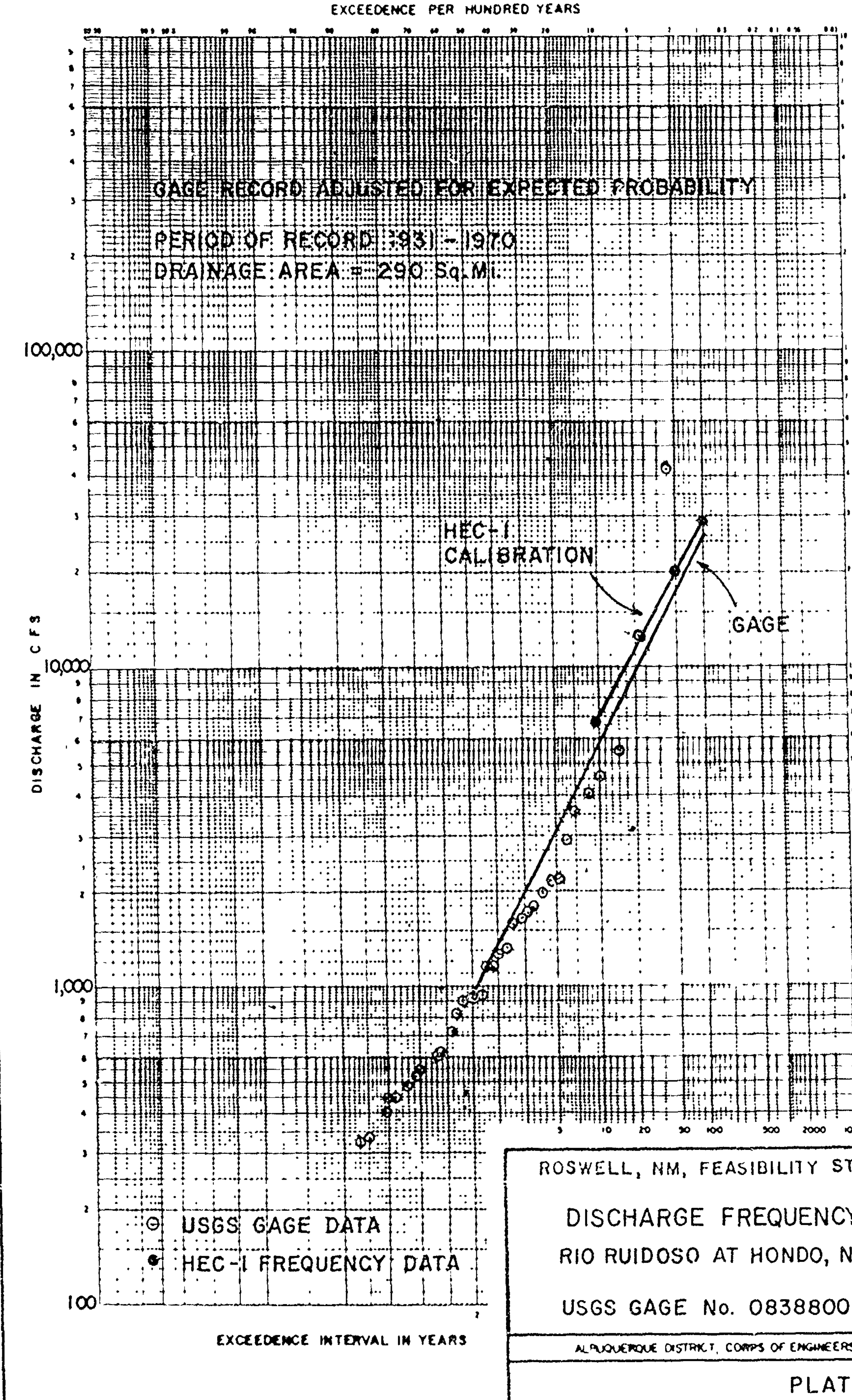
DISCHARGE FREQUENCY
RIO BONITO AT HONDO, NM

USGS GAGE No. 08389500

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 5

SWA 648
MAR 78



ROSWELL, NM, FEASIBILITY STUDY

DISCHARGE FREQUENCY

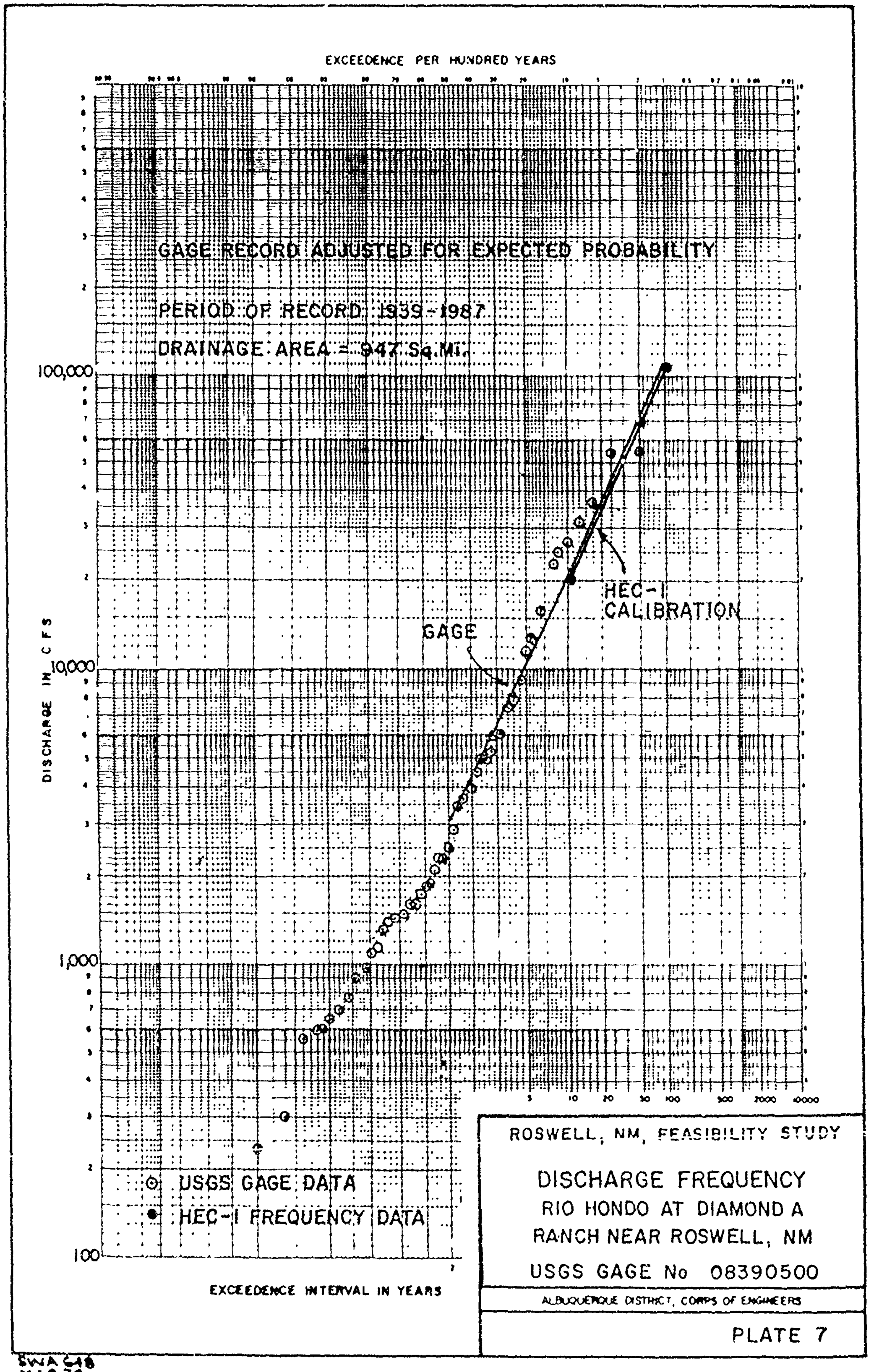
RIO RUIDOSO AT HONDO, NM

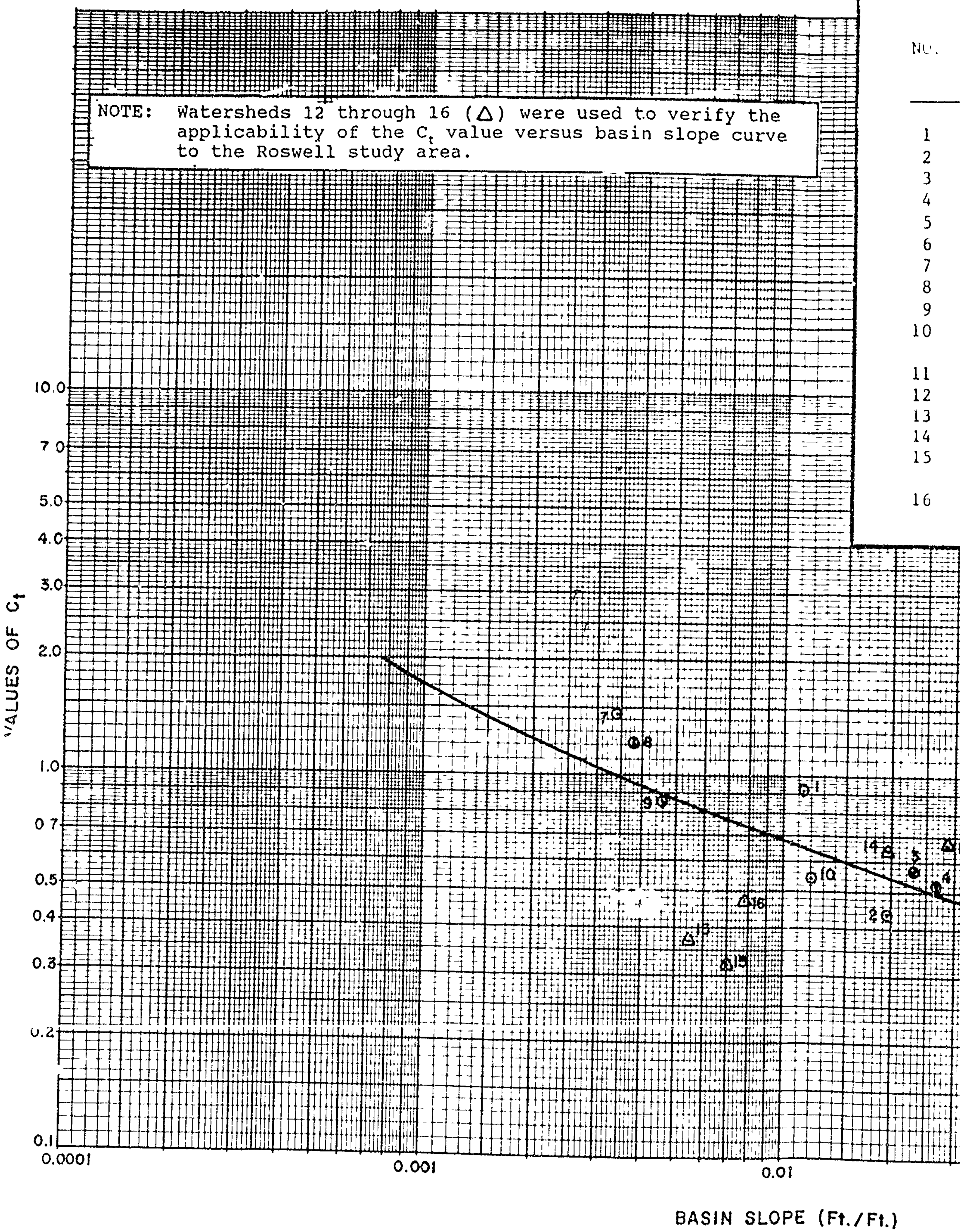
USGS GAGE No. 08388000

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 6

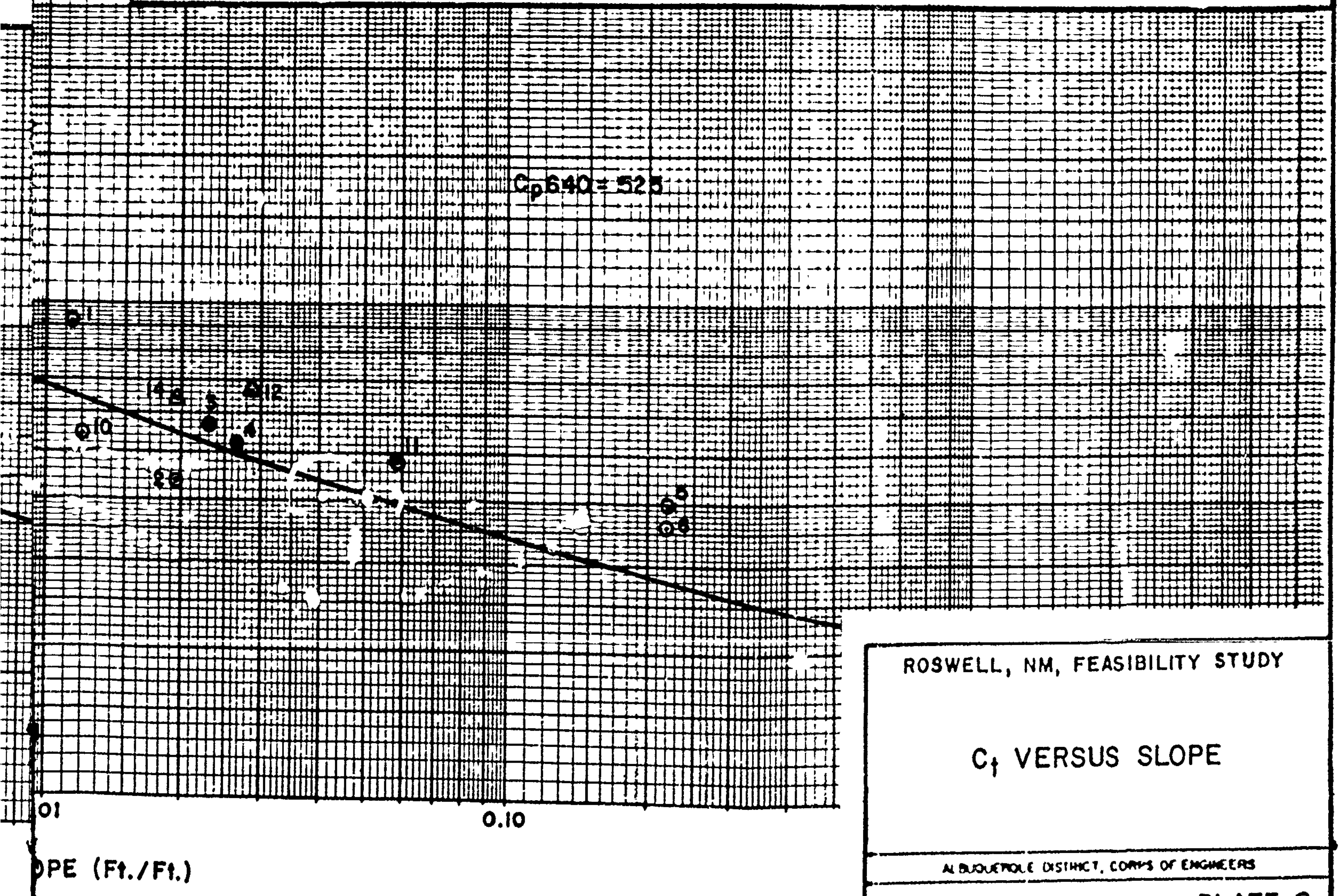
SWA 648
MAR 78


EXCEEDENCE PER HUNDRED YEARS
GAGE RECORD ADJUSTED FOR EXPECTED PROBABILITY
PERIOD OF RECORD 1939-1987
DRAINAGE AREA = 947 Sq. Mi.
DISCHARGE IN CFS
100,000
10,000
1,000
100
HEC-I CALIBRATION
GAGE
USGS GAGE DATA
HEC-I FREQUENCY DATA
EXCEEDENCE INTERVAL IN YEARS
ROSWELL, NM, FEASIBILITY STUDY
DISCHARGE FREQUENCY
RIO HONDO AT DIAMOND A
RANCH NEAR ROSWELL, NM
USGS GAGE No 08390500
ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS
PLATE 7
SWA 648
MAR 78



NOTE: Watersheds 12 through 16 (△) were used to verify the applicability of the C_t value versus basin slope curve to the Roswell study area.
VALUES OF C_t
10.0
7.0
5.0
4.0
3.0
2.0
1.0
0.7
0.5
0.4
0.3
0.2
0.1
0.0001
0.001
0.01
BASIN SLOPE (Ft./Ft.)
1
2
4
5
7
8
9
10
14
15
16
No.
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

NO.	NAME	DRAINAGE AREA (SQ.MI.)	SLOPE (FT./FT.)	C_t	C_p	$C_p 640$
1	Willow Creek above Heron Reservoir, NM	112.0	0.0113	0.92	0.85	542
2	Skunk Creek near Pheonix, AZ	64.6	0.0193	0.44	-	-
3	New River @ New River, AZ	85.7	0.0231	0.57	-	-
4	New River near Rock Springs, AZ	67.3	0.0267	0.52	-	-
5	Rio En Medio near Santa Fe, NM	0.63	0.2200	0.40	0.84	540
6	North Fork Tesuque Creek near Santa Fe	1.6	0.2200	0.36	0.84	540
7	Rio Puerco near Bernardo, NM	6078.0	0.0034	1.45	0.79	506
8	Rio Puerco @ Rio Puerco, NM	5455.0	0.0038	1.21	0.82	525
9	Arroyo Chico near Guadalupe, NM	1378.0	0.0046	0.86	0.81	518
10	Alamogordo Creek, Tributary of Pecos River	67.0	0.0120	0.54	0.79	506
11	Albuquerque, NM, Watershed W-II	0.063	0.0591	0.48	0.84	540
12	Rio Ruidoso at Hondo, NM	290.0	0.0290	0.66	0.88	561
13	Delaware River near Red Bluff, NM	356.0	0.0055	0.37	0.71	454
14	Rio Bonito at Hondo, NM	295.0	0.0194	0.64	0.89	570
15	Rio Felix at Old Highway Bridge at Hagerman, NM	791.0	0.0070	0.32	0.92	589
16	Salt Screwbean Draw near Orla, TX	501.0	0.0078	0.47	0.89	570

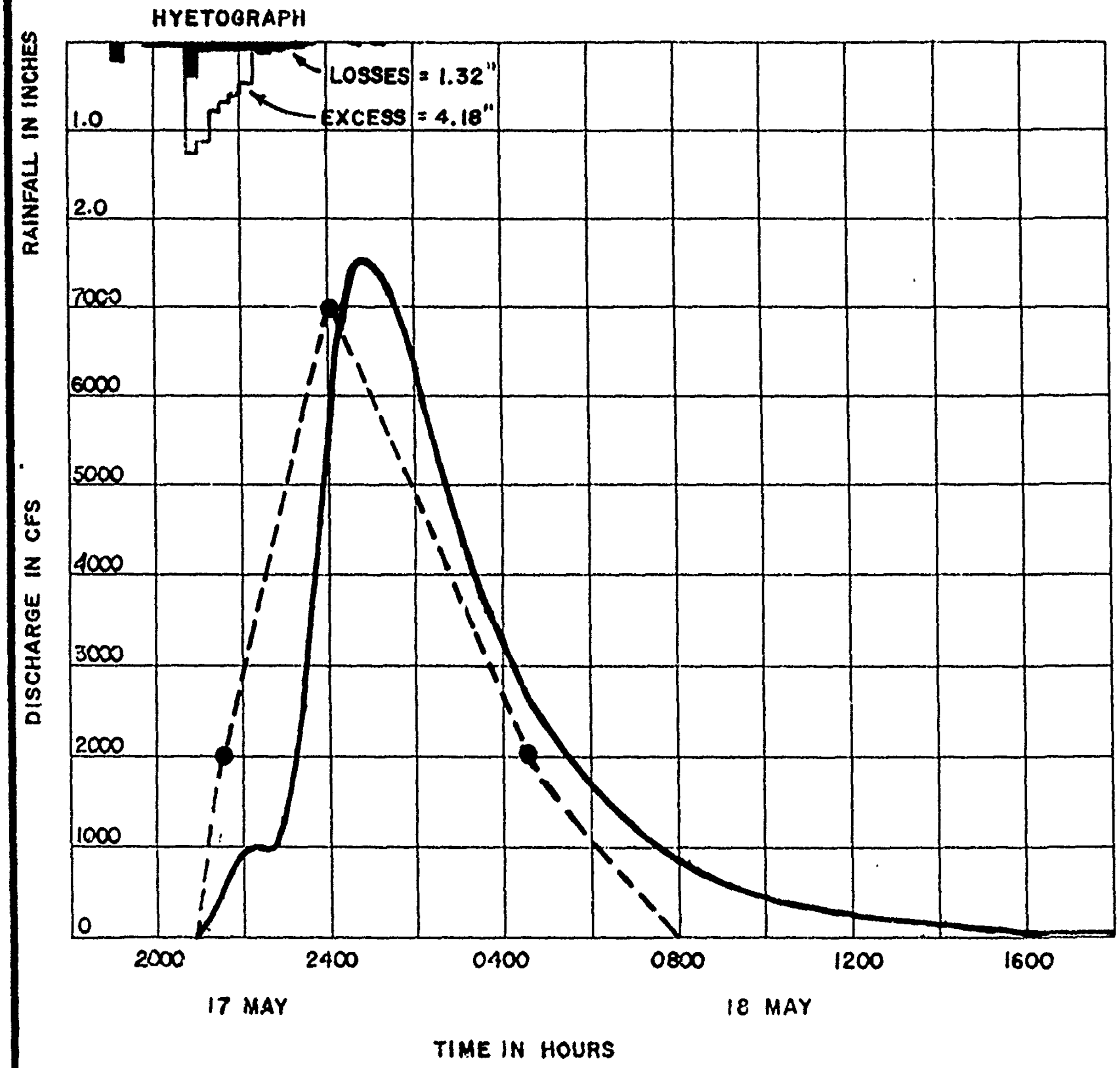


ROSWELL, NM, FEASIBILITY STUDY

C_t VERSUS SLOPE

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 8



OBSERVED FLOOD HYDROGRAPH
COMPUTED FLOOD HYDROGRAPH

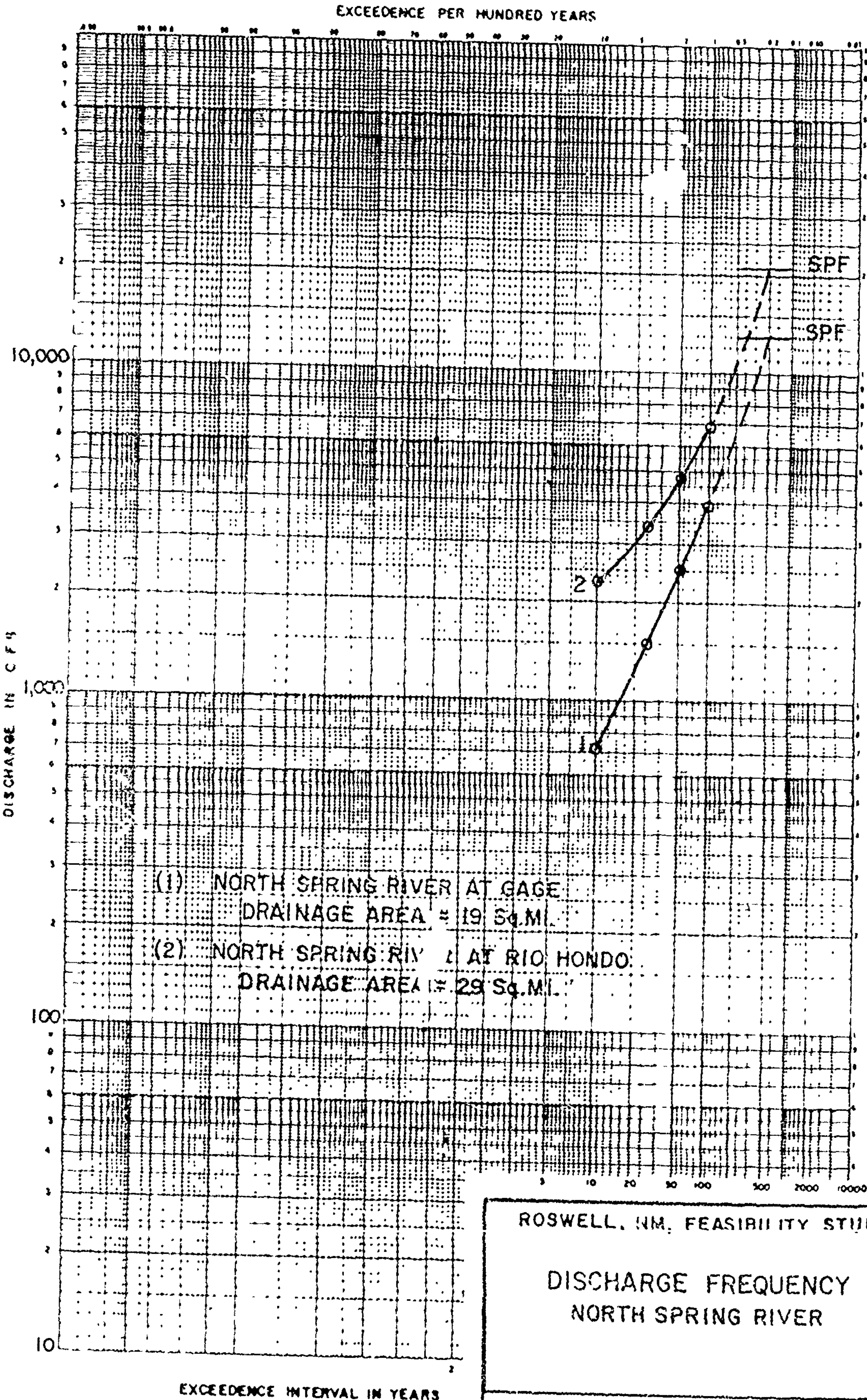
ROSWELL, NM, FEASIBILITY STUDY

STORM OF MAY 1954

NORTH SPRING RIVER
AT WYOMING STREET

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 9



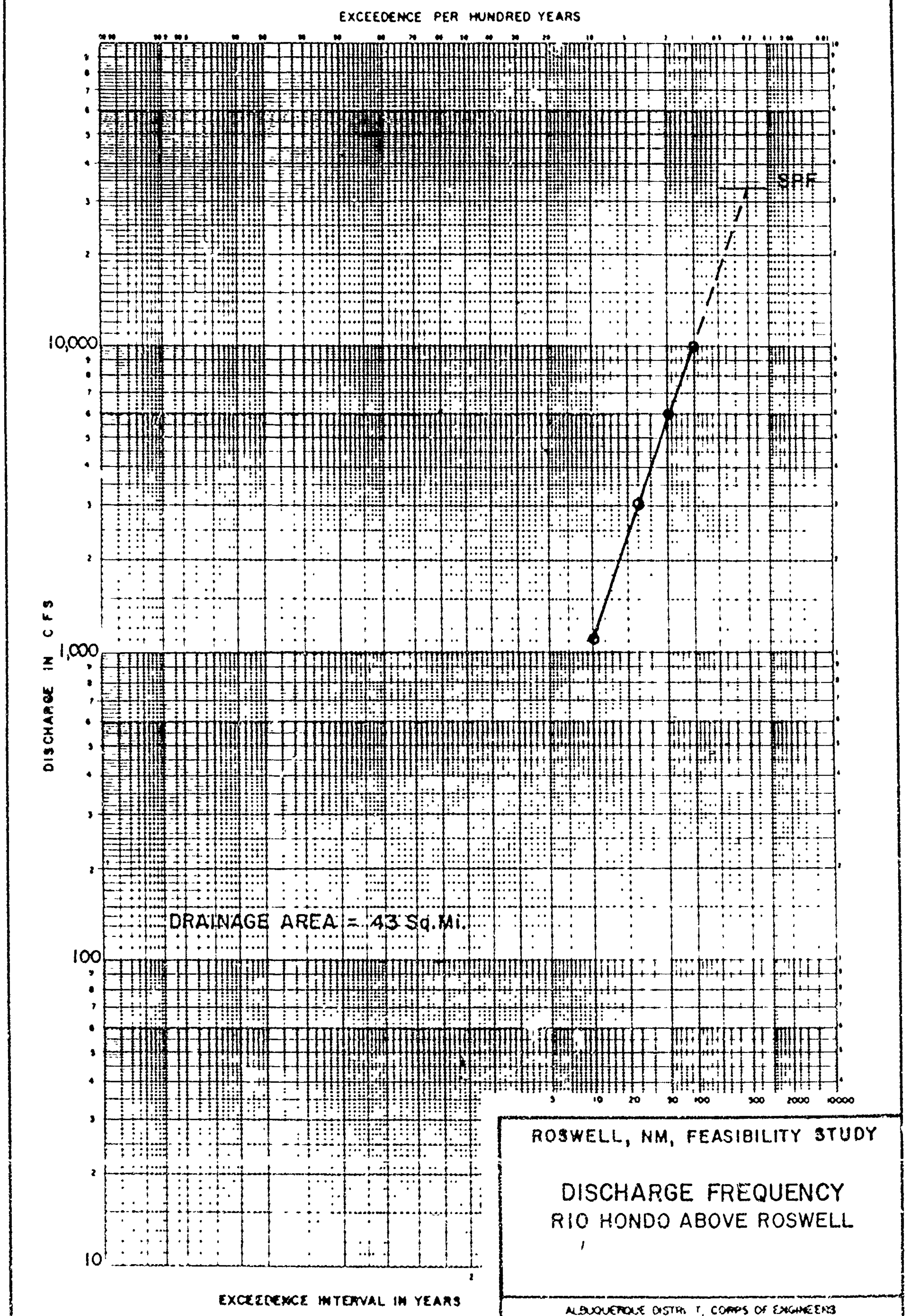
ROSWELL, NM, FEASIBILITY STUDY

DISCHARGE FREQUENCY
NORTH SPRING RIVER

ALBUQUERQUE DISTRICT CORPS OF ENGINEERS

PLATE 10

SWA 648
MAR 78



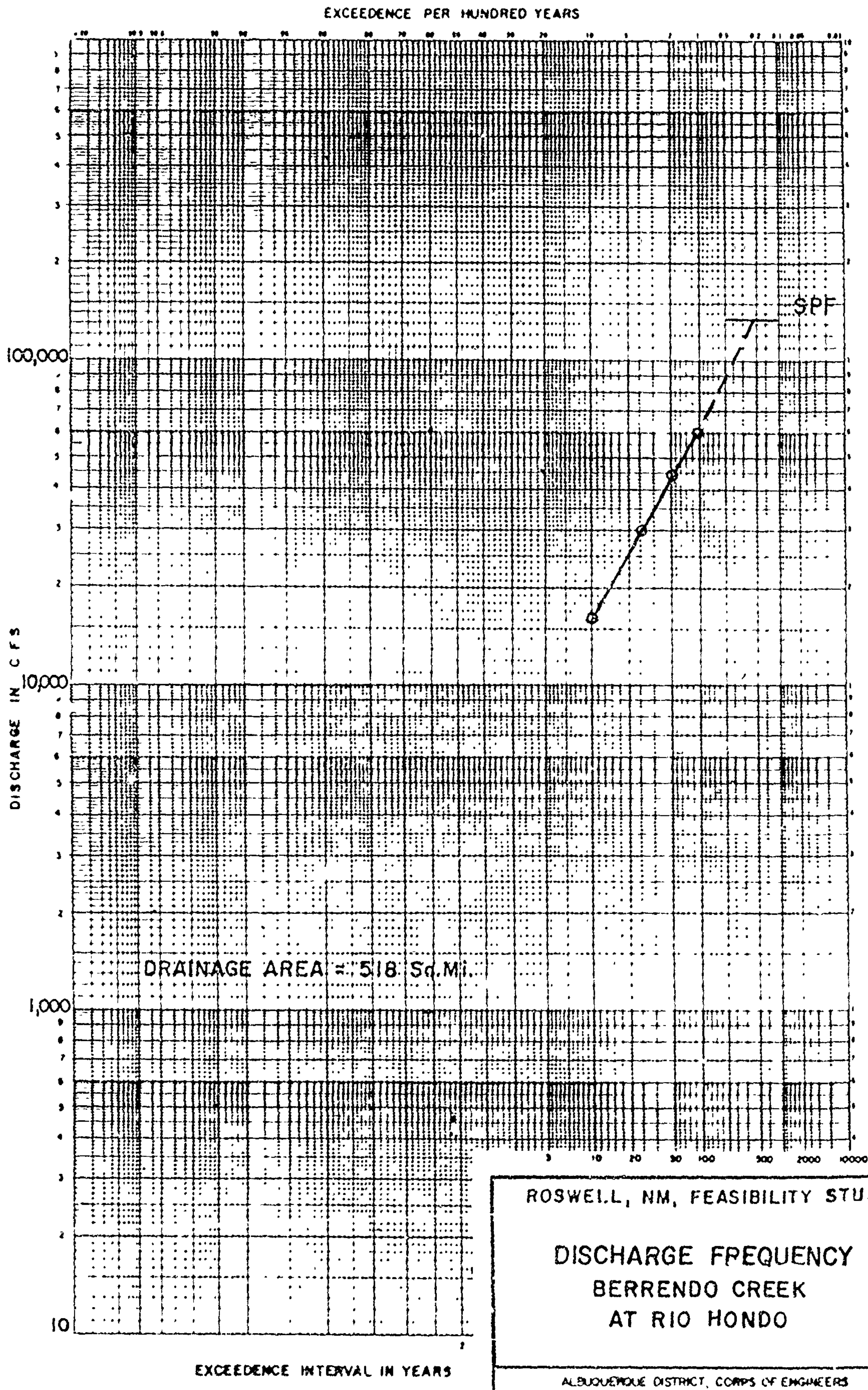
ROSWELL, NM, FEASIBILITY STUDY

DISCHARGE FREQUENCY
RIO HONDO ABOVE ROSWELL

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 11

SWA 648
MAR 78



ROSWELL, NM, FEASIBILITY STUDY

DISCHARGE FREQUENCY
BERRENDO CREEK
AT RIO HONDO

ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS

PLATE 12

SWA 648
MAR 78

Impacts of Gravel Pit Storage at Roswell, New Mexico

by

Olga Boberg

SUMMARY OF DISCUSSION BY LOREN W. POPE

There was considerable discussion of this paper mainly in regard to the design and operation of the gravel pits. Typical design questions are listed below.

(1) How deep is the gravel pit?

(2) How is the water removed from storage if it is below invert of the channel?

(3) Is there any benefit to ground water recharge?

(4) How was the storage handled in the existing conditions model?

(5) The upstream control section conceptually consists of a levee to force flow either into the channel or into the gravel pit storage, has any consideration been given to buying easements for the flooding induced by the levees?

Ms. Boberg stated that the design details had not been developed but that these details would be taken into account in the design of the gravel pit storage system. The basic concept was to store during a flood event and release through a controlled conduit after the flood waters receded.

Other general questions and responses.

(1) Have you considered a flood warning system? No

(2) Have any measures been taken to protect the storage areas and assure their availability for flood event? No, but these details will have to be worked out in the local agreements.

DROUGHT CONTINGENCY PLANNING

by

Cecil P. Davis[1]
and
James W. Stirling[2]

1. Introduction

a. **Study Purpose.** We in the South Atlantic Division of the Çorps of Engineers experienced an extended period of average rainfall for almost three decades prior to 1980. We had neither severe droughts nor major floods. As a result of this very moderate weather pattern, the public came to expect that Corps projects would generally meet all project purposes with only a minor impacts on any one purpose. During the 1980's we have experienced much larger deviations from this moderate weather, including the worst drought in the sixty-five year record. We realized a thorough review of our water management practices was appropriate. This paper gives an overview of that review relative to drought contingency planning and management for project purposes.

b. **Key Issues.** Several key issues have come to light as a result of our own review and the review of others such as the General Accounting Office (GAO). Some of these issues deal with the time required to appropriately study and develop drought contingency plans. Others deal with the project purposes and our authority to manage for those not specially listed in the authorizing legislation. Other issues have to do with management for a purpose that was specifically authorized but has no cost allocated to it.

c. **Summary of Findings.** It is certainly desirable to complete drought contingency studies prior to entering a drought. We had a plan for only one basin. Ultimately we found that the management would have been little different under the drought contingency plans that are now complete. It would have been better public relations to have had the plans completed before the drought. Our review of project purposes reveals authority to manage for the seven purposes, some of which were not specifically authorized. Reaching this conclusion forces us to make more choices regarding the trade-offs in management. However, it does result in the "greatest beneficial use" of the projects.

1/ Chief, Hydrology & Hydraulics Division, South Atlantic Division, US Army, Corps of Engineers
2/ Deputy Division Counsel, South Atlantic Division, US Army, Corps of Engineers

2. Physical Setting and Available Data.

a. Description of Projects.

The South Atlantic Division includes the area of the U.S. generally known as the Southeast. It includes large portions (or all) of MS, AL, FL, GA, SC, NC, and VA. Also, the division includes Puerto Rico which is in the Jacksonville District. The following map shows the SAD area.

The area is relatively rich in water resources and includes numerous Corps projects. In the Jacksonville District, the Central and Southern Florida (C&SF) project, which incorporates most of Florida south of Orlando, provides the main water supply for this area as well as providing significant flood control benefits. Water supply from this project and its operation in general, provide the backbone of municipal water supply in South Florida, water for vast areas of agriculture and for the Everglades National Park. The Corps also operates many other water resource projects in Florida.

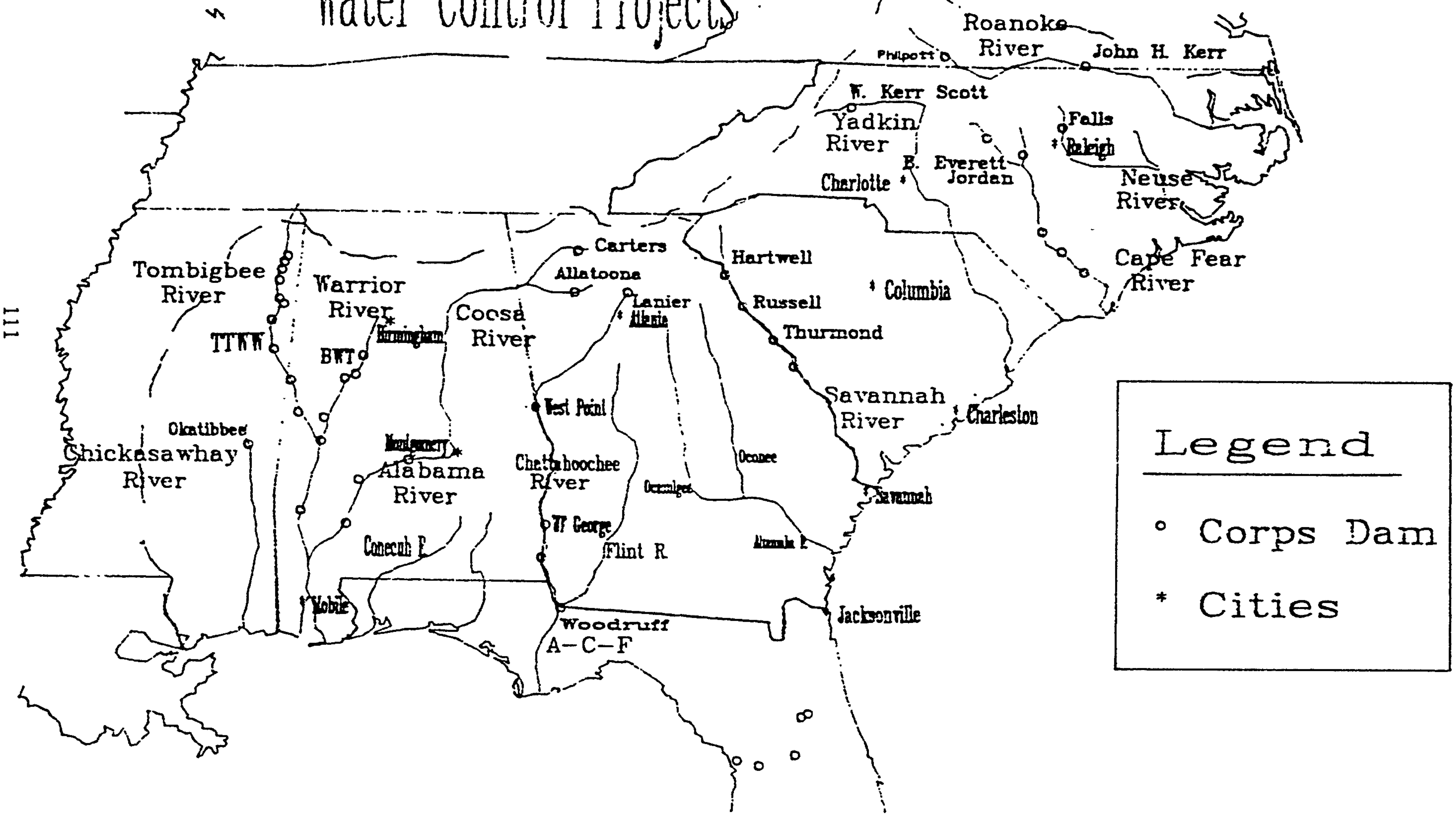
Alabama and Mississippi have numerous navigation-oriented projects to include the Tennessee-Tombigbee Waterway as well as the Black Warrior-Tombigbee Navigation System.

Alabama as well as Georgia, South Carolina, North Carolina and Virginia have several hydropower projects. These 13 projects have a capacity of about 2400 MW (normal capacity) which represents 5%-10% of the generating capacity within this area. This power is marketed by the Southeastern Power Administration (SEPA) to utilities throughout the area. Of the 13 hydropower projects, 10 contribute to one marketing arrangement known by SEPA as the Georgia- Alabama System. This marketing system includes projects in three different river basins. (Alabama, Apalachicola, Chattahoochee and Flint (ACF), Savannah).

b. Description of Available Pertinent Data.

Our projects are important for flood control, hydropower, navigation, recreation, water quality, water supply and fish and wildlife. Lanier is the main source of municipal water for metro Atlanta. Thurmond provides the necessary flow for Augusta, Georgia and cooling water for several plants. Also, Lanier, W. F. George, and West Point provide water to augment navigation flows on the Apalachicola River. There is a strong coalition of navigation interest trying to enhance the navigation on the Apalachicola River. Industries have located downstream of several of our projects and they benefit from the stream reregulation provided by them. Several of the projects in our area have tremendous recreation appeal. Lanier, Thurmond, Hartwell and W. F. George traditionally rank among the top ten Corps of Engineers projects in recreation visitation.

South Atlantic Division Water Control Projects



Many of these projects were planned in the late 1940's. To be specific Buford (Lanier), Allatoona, Hartwell, and Thurmond (formerly Clarks Hill) were planned and designed in the 1940's and early 1950's - before air conditioning. The project proponents did not realize that capacity would have the high value relative to energy that it has today. The "sold amount" of capacity exceeds normal design capacity (nameplate) by about 15%. Because of the high summer power peak caused by AC demand, the projects are marketed and operated differently than originally anticipated.

The "push pull" reaction among upstream/downstream users puts the operation of the projects in the forefront of numerous special interest concerns (particularly during droughts). Navigation interest desire dependable navigation depths; water quality interest demand an adequate release to maintain acceptable conditions; recreationists demand stable lake levels; municipal water supply proponents demand a dependable supply. Hydropower is a dichotomy in itself; water must be released to generate the contracted energy but lake levels must be maintained to provide the contracted power (capacity).

The Corps is put into the "honest broker" position. We must not only make rational, well-justified decisions, we must inform the public of our decisions and the reasons for them.

3. **Study Approach.**

a. **Procedures adopted.** Following the 1981 drought we reviewed our reservoir water management plans. This review indicated a high priority should be given to completing drought contingency plans in accordance with ER-1110-2-1941. There were two prime reasons why the Apalachicola, Chattahoochee and Flint Rivers (ACF) Basin was the first basin selected for the development of a drought management plan. The 1981 drought had indicated a strong need and there was a "308" comprehensive basin study underway that could provide funding. The initial funding was received in October 1984. An interim drought management plan was completed by the Mobile District in April 1985. The demonstrated benefits of the ACF plan prompted the Savannah District to complete a plan for the Savannah River in March 1989.

Table I shows the projects and their purposes in the ACF and Savannah Basins. Take note of Buford, just northeast of Atlanta. It is of major significance to Atlanta and represents concerns of the basin for the states of Alabama and Florida. The Atlanta metro area represents forty percent of Georgia's population and fifty four percent of the ACF Basin population. The Atlanta Regional Governments now have temporary authority for use of Lake Lanier Storage for Water Supply, pending completion of a storage reallocation study.

TABLE 1

PROJECT PURPOSES
FOR
CORPS PROJECTS IN SEPA'S GEORGIA-ALABAMA SYSTEM

Project	Authorizing Document	Purposes When Authorized	Other Purposes Added & Authority
Walter F. George	House Res. 5/19/53	NAV, POW	REC (FCA 1944 & PL 89-72) F&WL (PL-85-624)
West Point	HD 87-570 (FCA 1962)	NAV, POW, FC F&WL, REC	
Buford (Lanier)	HD 80-300 (RHA 1946)	NAV, POW, FC WQ, WS	REC (FAC 1944 & PL 89-72) F&WL (PL 85-624)
J. Strom Thurmond	HD 78-657 (FCA 1944)	NAV, POW, FC	REC (PL 99-662 WS (PL 85-500) F&WL (PL 99-662)
Hartwell	HD 78-657 (FCA 1950)	NAV, POW, FC	REC (FCA 1944 & PL 89-72) WS (PL 85-500) F&WL (PL 85-624)
Richard B. Russell	SD 89-52 (FAC 1966)	POW, FC, REC, F&WL	F&WL Mitigation (PL 99-662)

b. **Review Background.** The 1981 and 1986 droughts were only about a year long. They were quite severe for agriculture and generally recognized by the media as such. The 1987-90 drought in comparison started after an above average fall and spring rainfall which filled the projects. It is noted that for Lanier the drought (mathematically-prime flow) was from Oct 85 - early 1990. In November 1987 the Corps decided to reduce releases to those which provide the "energy to meet capacity" and also provided water supply. Because it was not recognized we were in another drought, some disagreed with our management. Recreational interests thought we were drawing the lakes too fast while hydropower and navigation interests wanted no restrictions on releases. The coordination and agreement that had occurred in 1986 was not forthcoming in 1987. As the drought progressed into 1988 and 1989, our conservation-oriented approach was vindicated. However, there were some who challenged our authority to operate for various purposes. Therefore we initiated a comprehensive review with a goal of reaching agreement as to project purposes and restrictions these purposes might impose.

c. **Key Issues.** The review and subsequent discussions involved the people within the South Atlantic Division Office and its districts involved in the management of projects. This included Engineering, Planning, Counsel, Public Affairs, Operations and the Executive Office. Many discussions centered on Lake Lanier, a key headwater reservoir for Atlanta water supply and other purposes. The authorization for Lake Lanier specifically recognized flood control, hydropower, navigation, water quality and water supply. There is some controversy among vested interests on the management for these purposes and the Corps authority to modify what was considered the authorized management plan. However, the greatest controversy surrounds the management plans for purposes which have been authorized under generalized legislation. These are recreation and fish and wildlife for Lanier.

Recreation authorization is founded in the general legislation of the 1944 Flood Control Act (FCA) (P.L. 78-534) and the 1965 Federal Water Project Recreation Act (P.L. 89-72). The legislative history of the 1944 FCA reveals that Congress considered its grant of authority to develop and operate park and recreational facilities as an "additional authorization" beyond those granted in project-specific legislation. The object of the 1944 FCA was to add recreation to the other purposes and to consider all the purposes when developing management plans that would make the greatest beneficial use of what might otherwise be flood waters.

It is clear from some project authorization documents that Congress authorized purposes not specifically addressed in the economic justification of those projects. Consistent with this, it is our view that the Corps has the authority to exercise its discretion to give recreation consideration with other purposes. Congress has not, in our view, dictated a hierarchy of project purposes. Rather, it has vested considerable discretion in the Secretary of the Army and the Chief of Engineers to operate the Corps water resources development projects to achieve the greatest public benefits consistent with broad Congressional authorization. Inherent in this is a responsibility to adjust operating methods to meet changing physical conditions and/or public needs. A key element in this discussion is that, should the exercise of this discretion lead to a decision to operate permanently for recreation in a manner which significantly and adversely impacts other purposes, a reallocation of storage space to reflect this decision would be required.

Note that the argument is not that we are required to treat recreation as a project purpose equivalent to the specifically authorized purposes but that (a) we have sufficient authority from Congress to operate our projects for recreational purposes and (b) how that authority is exercised is a matter involving considerable discretion on the part of the Secretary of the Army and the Chief of Engineers.

There has been much discussion to the effect that a purpose may not receive any consideration if no costs were allocated to that purpose. Projects formulated after passage of P.L. 89-72 in 1965 have costs allocated to recreation. That act provided the first statutory definition of federal interest and cost-sharing requirements for recreation at reservoir projects. Previously, no allocation of joint-use project costs to recreation was required. P.L. 89-72 was enacted several years after approval by President Kennedy of new standards and policies for development of water resources projects which first addressed principles for establishing recreation benefits, including those for the recreation aspects of fish and wildlife. These standards and policies are published as Senate Document 97, 87th Congress, 2d Session (May 29, 1962).

That no such standards existed when several of our projects were authorized does not mean that recreation cannot be a purpose of that project or must be regarded as inferior to other purposes. The 1944 Flood Control Act had been law for two years at the time of the Lake Lanier authorization, and was thus applicable to this project. Moreover, the authorizing document for Lake Lanier is clear in (a) considering recreation, water supply and water quality as purposes of the project and (b) allocating no costs to any of these purposes. It is particularly noteworthy that no costs were allocated to water supply since the operation described in the authorizing documents clearly require that some storage be used to satisfy the water need in the Atlanta area. The document also recognizes that this need will surely increase in the future. This document (House Document 80-300) reports that it was "impracticable" to determine a monetary value for recreation, but recognized the benefits to be "real and large". The subsequent cost allocation for this project was prepared in keeping with then-existing authorities and polices.

There is often an unfounded assumption that an operation for one purpose hurts other purposes, ie. that water management is a "zero sum" game. The conflict between holding water for recreation pools versus releasing for hydropower generation is often cited. However, conserving water can also increase hydropower benefits by protecting capacity. We are required to plan and operate projects for widespread benefits which is different than maximum revenues. Vested interest may have a revenue loss although benefits increase.

d. **Hydropower Discussion.** The value of hydropower is measured in two components, namely capacity (the ability to generate power) and energy (the quantity of power actually generated). In the Southeast area the monetary value of the capacity is 70% to 80% of the total hydropower value. The November 1987 decision to conserve water served to retain the maximum amounts of water within the reservoirs for as long a time as possible and thereby preserved a sufficient head which protected hydropower capacity in the Georgia-Alabama system. It was only at the very end of the drought that low reservoir levels reduced availability

below full capacity sold by SEPA. Had it not been for the conservation efforts begun in 1987, the reservoirs would have been too low for the marketing agency to meet contract capacity requirements. During this entire period, the water supply and water quality releases were made through the turbines, thereby assuring maximum hydropower benefits. The reduction in watershed runoff caused an approximate 50% shortfall in generation. This shortfall was due to the drought and not the actions of the Corps. No alternative method of operation could have produced "normal" hydropower over the course of this drought and any operation which produced more energy would have damaged the capacity function after end of the drought.

The current operation for hydropower is considerably different than that planned in the original authorizations. The authorizing documents envisioned a "base load" operation, where some hydropower plants would operate approximately 60% of the time. This base load was often based on reducing to prime flow (yield during period of record drought) when the pool level was one foot down into the conservation pool. By contract, and to the benefit of hydropower, the system has long been operated in a "peaking" mode. The use of a much shorter operating time associated with overloading the units yields a much larger hydropower capacity and thus greater hydropower benefits.

The 1949 Definite Project Report for Lanier specified that one of the operating criteria would be that only "prime power" would be generated when the reservoir water levels fell below the project guide curve. (Prime power, or primary energy, is a project's continuous energy output over the period of the most adverse flows on record or, in effect, on the basis of a then-known worst case scenario.) This Report computed prime power for Buford Dam as being about 2400 megawatt-hours per week. In reality, it has routinely been operated to generate well in excess of prime power during periods when Lake Lanier water levels were below the guide curve. The summer (July-September) pool level has historically been below the guide curve more than half the time, yet at the request of hydropower interests, the Corps generated about 3900 megawatt-hours per week during these times.

The 1949 report also did not differentiate between hydropower generation in summer or winter months. A relatively constant hydropower generation was used as the basis of project authorization. In response to changing needs and circumstances in society (greatly expanded use of air conditioning, as an example), this original operating plan has been altered considerably, so that power generation in the summer greatly exceeds that in the winter. Because this period of high generation coincides with the historic period of low rainfall and low inflows into these projects, lake levels are lower and the rate of drawdown higher in the summer months than they would have been under the original operating concept.

Changes to benefit hydropower have been made at other projects as well. For instance, the Millers Ferry and Robert F. Henry projects were constructed as "run of the river" projects without any ponding or pooling of water above the dams. This means that hydropower could be generated only to the extent that inflows were available at any point in time. In contrast to this, these projects have long been operated to retain water above the dams at night and release it during the day when the energy is more valuable.

These are among the operational changes within the Corps' discretion which, taken collectively, have dramatically increased the revenues and benefits of these projects to hydropower interests. There have been certain detriments to other authorized purposes. Whatever the equities of this may be, there has been no reallocation of project costs to hydropower as a result of these operational changes.

e. **Non-Specific Authorizations.** The authorizing documents for our reservoir projects generally recognized a large potential for recreation development, and Congress has provided authority to realize that potential. The "real and large" benefits envisioned decades ago have materialized at many of our South Atlantic Division projects. Five of these are among the ten most-visited Corps of Engineers projects in the United States. Our ability to make direct charges for these visits is extremely limited, however. We have authority under federal law to charge for camping and other specialized facilities and services. We are specifically prohibited, however, from charging day use fees or fees for use of such things as boat ramps. Congress has not been inclined in the past to support increased recreation use fee authorities. Beyond this, at P.L. 89-72 Projects, local sponsors are required to pay 50% of the cost of development of recreation facilities and 100% of the operation and maintenance cost for those facilities. The federal treasury also derives revenue from leases of project lands, such as marinas and similar service facilities. Among these various sources, we are collecting payment from recreation interests to the extent provided by current law.

Recreation interests clearly benefit from the existence of our multi-purpose projects. This was well understood at least as early as 1944; nevertheless, the legislative and administrative policies which have evolved have not sought to charge recreational interests for the cost of these projects. Because we do not operate any of the Georgia-Alabama projects specifically for recreation in any way which significantly and adversely affects other authorized purposes, there is no basis, in our view, for a reallocation. Should reallocation studies be conducted, they should address all project purposes and arrive at a completely new cost allocation reflecting current benefits, interest rates and construction price levels.

4. **Study Results.**

a. **Summary of Study Results.** A major goal of the ACF Drought Management Plan was to provide some indicators to define when a drought was in progress. The plan considered general information on stream flows, groundwater and soil moisture. Numeric indicators have provided minimal information on the droughts beginning.

The other major goal was to define action levels for each reservoir. Figure 1 shows these levels for Lake Lanier. The actions to be taken within each reservoir level are very general, with the intent being to make more specific decisions during a review of current conditions and within coordination meetings with other Federal, state and local interest groups.

The drought contingency plan for the Savannah River Basin was begun in Jan 87 and completed in Mar 89. This plan was managed by the Savannah District Planning Division. It developed and coordinated a typical planning document, including several reviews and public meetings. The plan has a series of reservoir pool guide elevations that trigger actions. These guides were set with consideration for hydropower needs, recreations needs and impact levels and also minimum releases necessary for water supply and streamflow requirements. The guides for Thurmond Lake are shown as Figure 2.

The Savannah River Drought Contingency Plan has a more definitive approach. Although it too has a goal for coordination with project users and beneficiaries, the plans define more specific actions based on reservoir levels.

b. **Specific Management.** The reservoir management for one purpose could well impact other purposes. However, as I mentioned earlier we often migrate toward an argument regarding recreation versus hydropower. Lets' suppose recreation was not originally specifically authorized and has no joint cost allocated to the purpose. Let's then ask whether we can utilize reservoir guides that do not contemplate reservoir drawdowns to the originally - planned maximum limits.

First, I point out that the minimum "power pool" line is one created purely as a matter of economics and physical capability of the power units. Indeed, the Definite Project Report for Lanier cited above describes elevation 1,030 - later changed to elevation 1,035 - as "the economic limit of drawdown of the Buford Power Pool". The Definite Project Report for the Thurmond Project likewise describes the 25 foot drawdown as being based on "a comparison of returns and cost of power generation". It does not necessarily follow from this that the area above this drawdown line is one reserved exclusively for power; it suggests only that power generation is economically viable above this level.

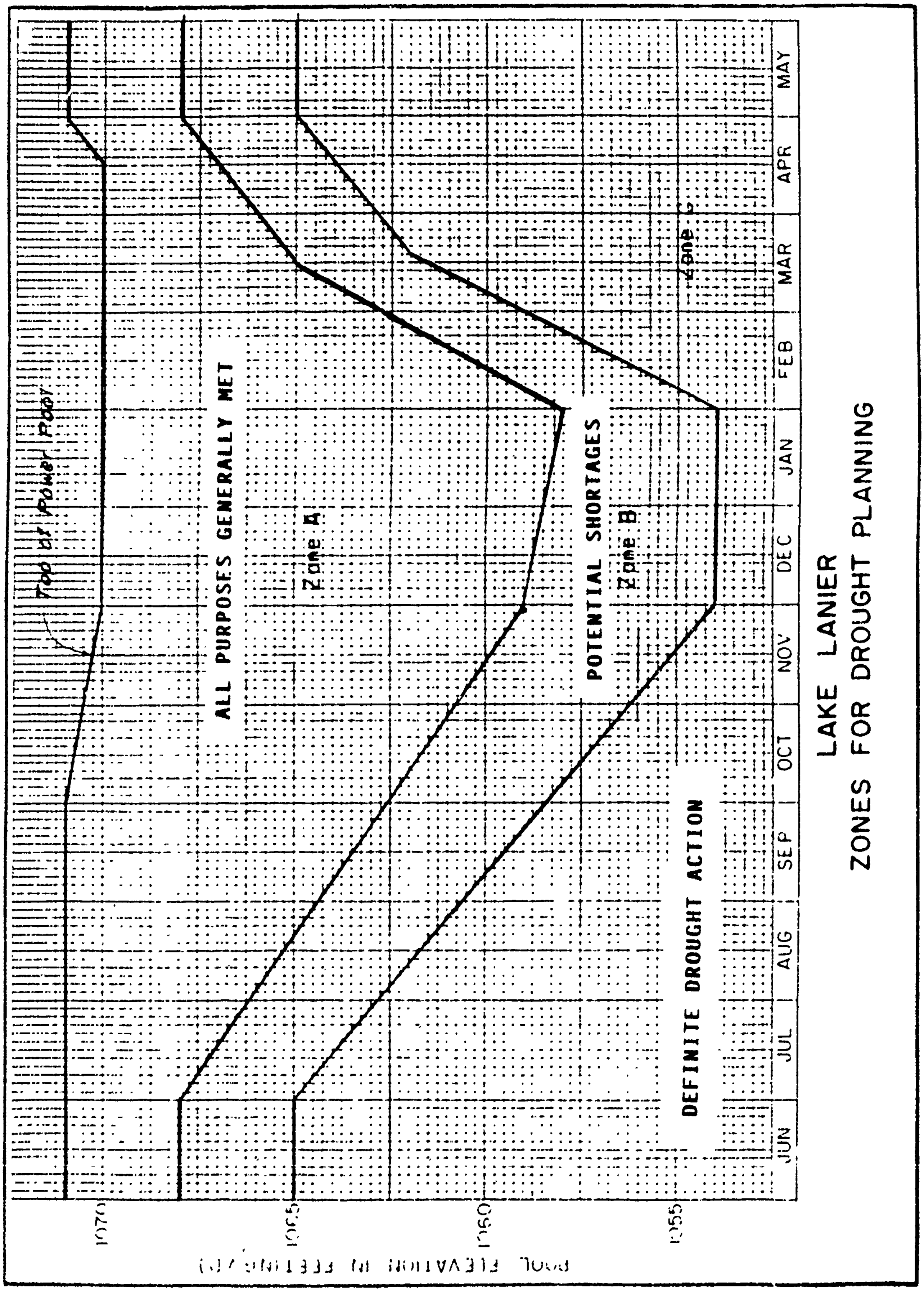

LAKE LANIER
ZONES FOR DROUGHT PLANNING
ALL PURPOSES GENERALLY MET
Zone A
POTENTIAL SHORTAGES
Zone B
DEFINITE DROUGHT ACTION
Top of Power Pool
JUN
JUL
AUG
SEP
OCT
NOV
DEC
JAN
FEB
MAR
APR
MAY
1070
1065
1060
1055
POOL ELEVATION IN FEET (NGVD)

Figure 1

Thurmond Reservoir Action Levels

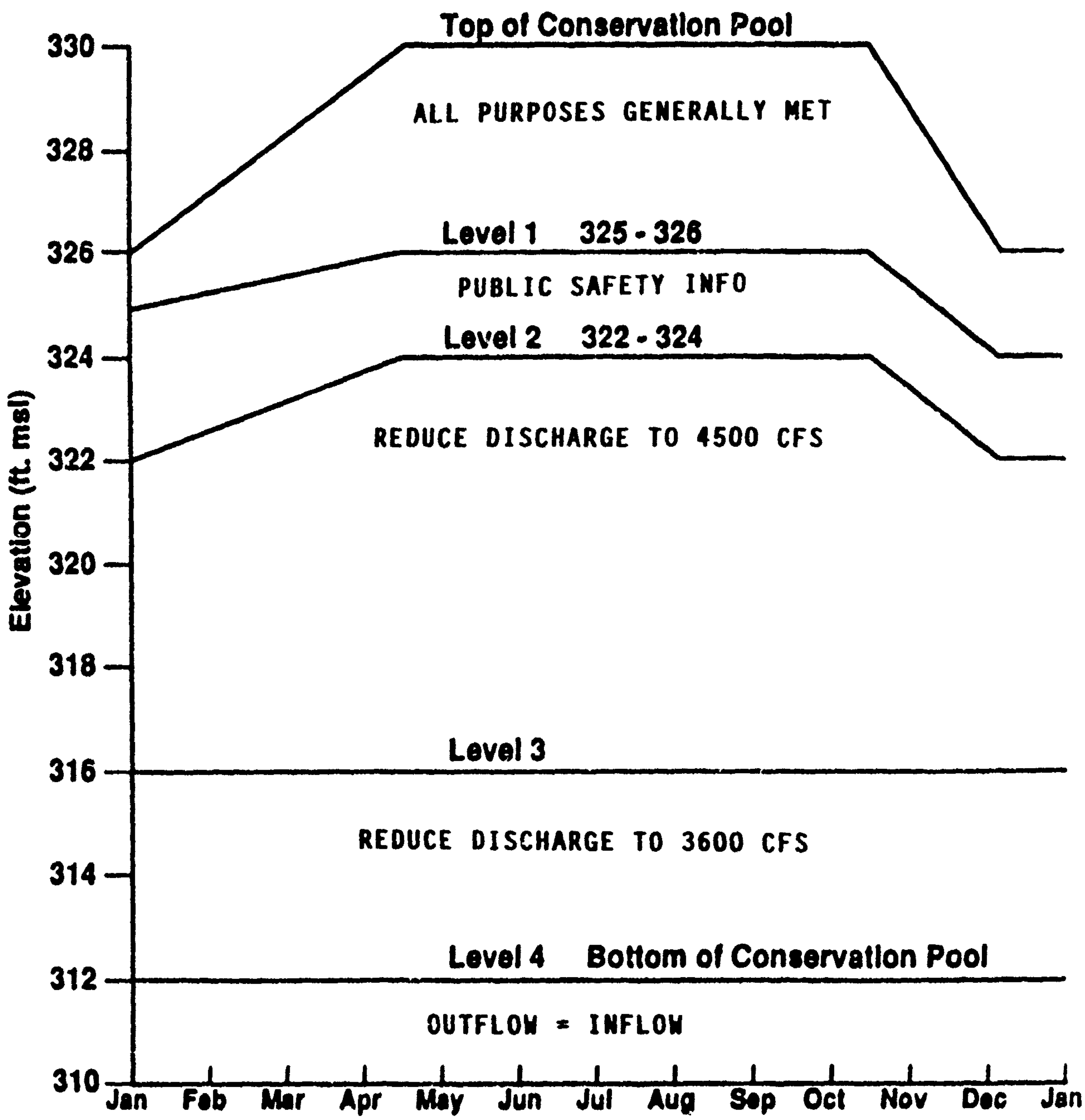


Figure 2

Decisions about how the Buford Project should be operated are ultimately a function of the authorized purposes of that project and the conditions existing at various times and under different circumstances. To accept that navigation, water supply and recreation are legitimately authorized purposes at the Buford Project means that these purposes must have some claim to the reservoir operating pool. Power generation must necessarily be only one claimant on this pool.

Other project purposes aside, a consideration of operation for power alone still suggests no improprieties in a guide curve change, particularly if this is done to protect the project's dependable capacity. Given the relative values of capacity and energy, an operating policy which gives greater weight to protection of capacity than to generating to the bottom of the pool makes practical sense and should be well within our discretion. This is suggested in the July 31, 1985 version of EM 111-0-2-1701, which addresses power operation in times of adverse flows. This states, at page 5-93:

> "Because the rule curve is based on the most adverse sequence of flows in the period of record, the project can be operated through the period of record without any failure to meet firm energy requirements or any violation of the minimum power pool. However, in actual operation, there is always the possibility that a more adverse sequence of flows will occur. Hence, if an extended period of low flows occurs, and the reservoir falls well below the rule curve, contingency measures would likely be taken to conserve the remaining storage. First, attempts might be made to purchase thermal generation to help meet the firm energy requirement. If this is not enough, opportunities for reducing firm load would then be examined". (Emphasis added)

A guide curve is devised to indicate operation of a reservoir to obtain best results based on past experience. The curve is a result of an operating plan; as such, so long as the curve accurately reflects the plan, it is really the plan which should be subject to scrutiny. If this operating plan is (a) within our authority and (b) a reasonable exercise of that authority, we should feel comfortable accepting the plan as an operating guide.

Lastly, it is worth stating that allocation of a portion of the project cost to a specific purpose does not give rise to some property interest in the reservoir. Also, all the project purpose proponents, whether they make a payment or not, receive far greater benefits than anticipated and none pay fair market value for the products provided by the project. The statutory authorities of the Corps and SEPA make the Corps the project manager and decision maker about project operations and priorities. The Corps is obligated to solicit information from project users so as to

ensure the advantages and disadvantages of each alternative operation are given adequate consideration. Thus, as we operate rationally within the range of purposes and authorities provided by law, operational changes and resultant alteration of guide curves are reasonable and within the general guidance provided by statutes.

5. **Conclusions**

a. **Project Performance.** The projects were managed in a ways to achieve large benefits for all the purposes during the recent droughts. The South Atlantic Division of the Corps recognized the desirability of drought contingency plans and are pursuing completion of them for all projects. The plans should be definitive yet retain sufficient flexibility to incorporate available information on existing conditions into water management decisions. There are real needs to improve data analysis and presentations of information so as to make information more readily available to Corps personnel and also to users.

b. **Hindsight Observation.** The Corps has been given considerable discretion authority in evaluating its projects and modifying their management plans to provide beneficial use of project resources. In fulfilling this responsibility the Corps should coordinate with all users and adequately describe the management plans and be prepared to justify its rational for its decisions. There were sound reasons why we did not have all drought contingency plans completed. However, it would have made management much easier had they been completed. The public is generally willing to hear information about conflicts in water use. Our review confirms that the South Atlantic Division has traditionally based its water management plans on the current public needs, consistent with the project authorization. We have considerable latitude in managing the projects and procedures to reallocate storage and costs if necessary.

Drought Contingency Planning

by

Cecil P. Davis

SUMMARY OF DISCUSSION BY LOREN W. POPE

Questions & Responses

Was this all O&M funded? Mr. Davis stated that it was all O&M, however they did have a Section 308 study in the Savannah River Basin.

Did you have any problems getting funds? No we didn't but the DCP program had Division and OCE attention which was very helpful. Dick DiBuono, HQUSACE, added that we had been able to use the GAO audit to assist in getting funds for this project.

What role did the states play? We formed a committee of Georgia, Alabama, and Florida. We thought that it would be a voting committee; however, it didn't workout very well as it became an open meeting to the press. It was therefore changed to an information exchange type of committee. SAD didn't think it would be very efficient to have an advisory board type of committee.

Comment by Shapur Zanganeh. He agrees that we don't have to abide with the power marketing agency on use of the projects. However, the design was based on the most severe drought of record and the CORPS is obligated to operate the project to provide the outputs that were indicated.

Response by Mr. Davis. The drought was more severe than the design drought and thus one would expect to have considerable problems in meeting design hydropower loads. Mr. Davis also emphasized that we were not operating the hydropower project as originally designed due to changing hydropower needs.

REALLOCATION IMPACTS ON HYDROPOWER AT TEXOMA

by

Ralph R. Hight[1]

Introduction

Study Purpose. A reallocation study of Denison Dam (Lake Texoma) was conducted in 1985 to develop information required by paragraph 7-3b of ER 1105-2-20 to reassign 77,400 acre-feet of power storage in Denison Dam (Lake Texoma) to satisfy the municipal and industrial water supply needs of the North Texas Municipal Water District (75,000 acre-feet) plus providing 2,400 acre-feet for future potential water supply users. Reallocation of 50,000 acre-feet of storage space was accomplished at this project under the discretionary authority of the Chief of Engineers in August 1983. Also, reserved in this project is 22,600 acre-feet of storage space for use by the City of Sherman, Texas, authorized by Public Law 85-146. The 1985 study was documented in a report entitled "Letter Report, Denison Dam (Lake Texoma), North Texas Municipal Water District" (Tulsa District Corps of Engineers, 1985). This report addressed the impacts of the total reallocation (150,000 acre-feet) on the project to ascertain if the last added increment of water supply seriously affected the purpose for which the project was constructed or if major structural or operational changes would be necessary. A significant change in the project could only be approved by Congress (Sec. 301(d) of Public Law 85-500, as amended).

Key Issues.

1) Initially the primary i[illegible] involved in the reallocation was the impact on hydropower outputs. Specifically, the amount of financial credit to be received by the preference customers because of the uniqueness of the power contract.

2) A lesser issue involved the Secretary's authority to approve a reallocation that would bring the total reallocated storage to 150,000 acre-feet. This issue was raised informally by the Southwestern Power Administration and likely was not a serious concern.

3) A major issue that arose following the reallocation approval and signing of the water supply contract involved the environmental impacts on Lake Texoma and on Lake Lavon where the water was diverted prior to distribution. This issue was raised through the Section 404 permit process for the Texoma intake structure and resulted in litigation initiated by the Oklahoma Wildlife Federation. The litigation resulted in a favorable court decision for the Corps and never actually impacted hydropower or the quantity of water supply storage reallocated.

Summary. Reallocation of an additional 77,400 acre-feet of hydropower storage to water supply in Lake Texoma would impact the average annual and firm energy from the project. Dependable capacity of the power plant would not be affected since the reservoir would continue to be operated as joint-use storage based on critical period analysis.

The reallocation brought the total storage reallocated from hydropower to 150,000 acre-feet but was determined to be within the discretionary authority of the Secretary of the Army.

Traditional credits to the PMA for revenues foregone plus appropriate adjustments in annual O&M charges were not acceptable in this case because of unique circumstances involving the

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[1]Chief of Hydrologic Engineering, Tulsa District, U.S. Army Corps of Engineers

power sales contract. Credits to the PMA were determined based on claims of potential losses by the two electric cooperatives (preference customers) having contracts for the total outputs of the Denison project. The PMA then negotiated actual credits to the co-ops. Essentially, those credits amounted to new thermal replacement value plus an assumed automatic escalation of five percent (5%) per year. The preference customers claimed they were entitled to the credits (thermal replacement) because they had contracted for all the outputs from the Denison project from the Government (PMA) and then that same Government (Corps) had removed some of those outputs and exposed them to financial loss at the hands of their power supplier.

Background

Denison Dam (Lake Texoma) was authorized for flood control and power in an Act of Congress approved 28 June 1938. Subsequent Acts provided for improving navigation, regulating flows of the Red River, and other beneficial uses. One of the Acts approved 14 August 1957 (Public Law 85-146, 85th Congress) authorized the Government to contract with the city of Sherman, Texas, for the use of not to exceed 41,000 acre-feet of storage space in Lake Texoma, for the purpose of providing the city a regulated water supply in an amount not to exceed 15,000 acre-feet per year. That storage yield relationship was based on providing water supply storage in the flood pool and maintaining the same hydropower firm energy during the drought of record. Water stored in the flood control pool for water supply was to be evacuated prior to the development of a flood. Since this is no longer considered a practical operation, a new study based on an integrated power and water supply pool between elevations 590.0 and 617.0 was made using projected lake storages for year 2044. This study indicated that 150,000 acre-feet of storage would provide a yield of 168,000 acre-feet per year or 150 million gallons per day during the drought of record. This change to the project falls outside the current policy limits of the Chief of Engineers to approve. These limits are 15 percentum of the total storage capacity allocated to all project purposes (15% X 3,338,000 acre-feet = 500,000 acre-feet) or 50,000 acre-feet, whichever is less. In analyzing the impacts of the proposed reallocation, we found that the reallocation would neither seriously affect the purposes for which the project was constructed nor would it involve major structural or operational changes. We, therefore, concluded the proposed reallocation could be implemented subject to the approval of the Secretary of the Army under the authority provided by the Water Supply Act of 1958. Upon this approval, 77,400 acre-feet of storage could be made available pursuant to the Water Supply Act of 1958 to the North Texas Municipal Water District and other potential water supply users.

Pertinent Hydrologic Data

Pertinent data for the project are provided in the following table:

Feature	Elevation (ft)	Acre (ac)	Capacity (ac-ft)	Equivalent Runoff (1) (in)
Top of dam	670.0	----	----	----
Top of flood control pool	640.0	144,000	5,312,300 (2)	2.51
Top of power pool	617.0	88,000	2,643,300 (2)	1.25
Bottom of power pool	590.0	43,100	1,031,300	0.49
Power storage	590.0-617.0	----	1,612,000	0.76
Flood control storage	617.0-640.0	----	2,669,000	1.26

Note: Data based on 1969 sedimentation survey

(1) From 39,719 sq mi of drainage area upstream from dam, 33,783 sq mi contributing
(2) Excludes inactive storage in Cumberland pool

Study Approach

Reallocation of Flood Control Storage. Consideration was given to raising the top of the conservation pool and reassigning 77,400 acre-feet of storage from flood control to water supply. That plan would result in a negligible reduction in flood control benefits. However, raising the top of the conservation pool could cause problems with fish and wildlife interests because of the effects on two wildlife refuges; adversely affect recreational facilities; and possibly require an Environmental Impact Statement. LMVD and the State of Louisiana have formally protested any reduction in the flood control capability of Lake Texoma (LMVD letter to SWD, 5 June 1974) and (State of Louisiana letter, 10 July 1973). Taking storage from the flood control pool as previously contemplated was based on evacuating the space prior to anticipated flood inflows and is not operationally practical. Therefore, in view of the above, no further consideration was given to reallocation of flood control storage.

Reallocation of Hydropower Storage. Consideration was given to reallocating an additional 77,400 acre-feet from hydropower storage to municipal and industrial water supply. The cost to the non-Federal interests for the reallocated storage would be established as the higher of either benefits or revenues foregone, replacement costs, or the cost of the storage in the Federal project as presented below.

Evaluation Data and Criteria.

Initiation of Project Construction	August 1939
Closure	October 1943
Available for Flood Control	January 1944
In-service Date	
1st Unit - 35,000 kw	March 1945
2nd Unit - 35,000 kw	September 1949

Storage after 100 years of sedimentation
(1944 + 100 years = 2044) will be 3,338,000 Ac-Ft between
top Flood Control Pool - Elev. 640.0 and
bottom Conservation pool - Elev. 590.0

Storage to be Reallocated to Water Supply 77,400 Ac-Ft
Hydropower Economic Life - 100 years
New period of Analysis
1944 + 100 years of sediment = 2044 - 1985 = 59 years

FY 1985 formulation interest rate was 8-5/8 percent

FY 1985 Water Supply repayment interest rate was 10.898 percent

Study Results

It was estimated that 77,400 acre-feet of storage in the hydropower pool between elevations 590.0 and 617.0 National Geodetic Vertical Datum (NGVD) would yield 86,700 acre-feet per year, or 77.4 million gallons per day during the critical drought period from July 1938 to March

1940 based on estimated storage for the year 2044, and assuming the storage received its proportional share of inflow. Existing installed capacity (nameplate) is 70,000 kilowatts with 54,000 kilowatts dependable at the bottom of power pool, elevation 590.0 NGVD. Minimum capability during the June through September peak power demand season is 67,300 kilowatts. The proposed reallocation would not impact the dependable capacity, although the associated firm energy and average annual energy will be reduced as shown in the following table.

Water Supply Storage (ac-ft)	Sediment Condition Year	Installed Capacity (MW)	Dependable Capacity (MW)	Firm Energy (GWH)	Avg.Annual Energy (GWH)
72,600	1985	70	54	95.4	214.4
72,600	2044	70	54	80.3	214.2
150,000	1985	70	54	88.8	207.6
150,000	2044	70	54	73.5	207.6

The average loss in kilowatt hours per year resulting from the proposed 77,400 acre-feet reallocation is 6,800,000 kwh based on 1985 sediment conditions and 6,600,000 kwh based on 2044 sediment conditions. These impacts amount to reductions of 3.2 percent and 3.1 percent, respectively, or an average loss of 6,700,000 kwh per year over the remaining economic life of the project. The most likely source of replacement power would be coal-fired generation based on FERC letter, May 25, 1984.

Hydropower Benefits Foregone. The loss of project benefits that would result from the reallocation of 77,400 acre-feet of storage were computed on the basis of existing price levels, interest rates, and conditions projected for the remaining economic life of the project. The benefits foregone from the lost power (6,700,000 kwh per year average annual) were estimated at $249,240 per year assuming Federal financing at a 8-5/8 percent interest rate and an energy value of 37.2 mills per kilowatt-hour (FERC letter, May 25, 1984). This loss of power was based upon the most likely alternative to be constructed. The alternative would be a coal fired plant due to the Denison plant factor being 34.9 percent. The present worth of the benefits foregone for reassignment of 77,400 acre-feet were based on the reduction in annual firm energy and determined in the following manner.

($249,240) (11.50622(1)) = $2,868,000

(1) Present worth factor for the remaining hydropower economic life of 59 years at a 8-5/8 percent interest rate.

Hydropower Revenue Foregone. The hydropower revenues that would be lost because of the storage reassignment were evaluated on the basis of existing rate levels and projected over the new period of analysis. The value of lost power based on existing SWPA average system rate of 17 mills per kilowatt-hour would be $113,900 per year (SWPA letter, June 22, 1984). This data was based on energy loss only since there would be no change in dependable capacity. The present worth of the hydropower revenues lost because of the conversion of 77,400 acre-feet were developed in the following manner.

Estimated average annual revenue loss 6.7 gwh @ 17 mills/kwh	=	$113,900
Less annual operation and maintenance (77,400/3,338,000) = 2.3188% X ($2,908,084) (FY 84))	=	67,400
Less annual major replacement cost (77,400/3,338,000) = 2.3188% X ($ 0 (FY 84))	=	0
		$ 46,500
$46,500 X 11.50622 (1)	=	$535,000

(1) Present value of an annuity factor for the remainder of the new 59 years period of analysis at a 8-5/8 percent interest rate.

Replacement costs. The Cooperatives (preference customers) indicated their loss would average $233,777 per year through 1990 and average $487,306 from 1990 until 2004. They assumed a 5% per year inflation factor and further assumed that after 1990 there would be a 1.1 MW capacity loss (penalty). The Cooperatives are dependent upon Texas Utilities Electric Company (TUEC) to schedule and deliver power and energy from Denison Dam to their service area. They have no contract with TUEC past 1990 but assumed the terms of the future agreement would be such that the reallocation of 77,400 acre-feet of storage would result in the capacity loss 1.1 MW (Southern Engineering letter, March 12, 1985).

Value of 77,400 acre-feet of storage. The value of the reallocated storage was determined by first computing the cost at the time of construction by using the Use of Facilities cost allocation procedure as follows:

$$\frac{\text{(Project joint-use construction cost)}}{\text{Total Usable Storage (ac-ft)}} \times \text{Storage reallocated (ac-ft)}$$

The cost allocated to the storage on this basis was then escalated to existing price levels by use of the Engineering News Record Construction Index. The updating factor was based on the index at the midpoint of the physical construction period as compared to the index at the beginning of the fiscal year in which the contract for the reallocation storage was approved. Computations to determine the value of the reallocated storage follows:

($45,810,877) 77,400 ac-ft/3,338,000 ac-ft = $1,062,200

Midpoint of construction period - September 1941

$$\text{ENR Index } \frac{\text{1 Oct 84}}{\text{Sept 41}} = \frac{4161}{263} = 15.8$$

Updated storage value
$1,062,200 X 15.8 = $16,784,000

Since the present value of the 77,400 acre-feet of storage was considerably more than the present worth of hydropower benefits or revenues foregone, water supply storage contracts approved in FY 1985 were based on a storage cost for 77,400 acre-feet of $16,784,000. Costs for contracts not approved in FY 1985 would be determined in the above manner, using data applicable for the fiscal year each contract is approved.

Economic Feasibility. As a test of economic feasibility, the annual cost of storage derived by the cost updating method was compared to the annual cost of the most likely, least costly, alternative that would provide an equivalent quality and quantity of water which the local interest would undertake in absence of utilizing the Federal project. To permit comparison, costs to obtain the water from Lake Texoma presented in Table 1 are expressed as an annual charge using a 8-5/8 percent interest amortization, plus annual operation, maintenance, and major replacement costs for the storage, treatment, and conveyance facilities.

TABLE 1

77.4 MGD FROM LAKE TEXOMA
2.4 MGD TO SHERMAN
75 MGD TO LAVON LAKE

	Capital Cost $ (1)	Annual O&M Cost $	Total Annual Cost $
Storage	16,784,000	67,000	1,526,000
Intake	2,830,000	21,000	267,000
Pipeline	32,702,000	58,000	2,900,000
Pumps	11,736,000	2,317,000	3,337,000
Treatment	17,609,000	1,684,000	3,214,000
TOTAL			11,244,000

(1) Costs amortized over a 59-year period, the remaining economic life of Lake Texoma, at 8-5/8 percent interest rate (Factor 0.08691)

Least Costly Alternative. A potential project, New Bonham Lake, located about 35 miles east of the city of Sherman, with an estimated dependable yield of about 89 mgd, was considered as a possible alternate water supply source of 77.4 mgd. The 75 mgd required by the North Texas Municipal Water District will be discharged into a tributary of Lavon Lake with the city of Sherman considered as the centrally located delivery point for the remaining 2.4 mgd. The estimated annual cost of $12,803,000 for the treatment and conveyance facilities, as well as the cost of the storage shown in Table 2, for New Bonham water is approximately $1,559,000 more than the estimated annual cost of $11,244,000 for the Lake Texoma water. Also, the New Bonham Lake project is only in a planning stage and it would be several years before water would become available from it.

It is apparent that Lake Texoma is the most viable potential source of the amount of water required to satisfy existing and short range future municipal and industrial water supply needs of this magnitude.

TABLE 2

77.4 MGD FROM NEW BONHAM LAKE
2.4 MGD TO SHERMAN
75 MGD TO LAVON LAKE

	Capital Cost $ (2)	Annual O&M Cost $	Total Annual Cost $
Storage	45,187,000	452,000	4,350,000
Intake	1,670,000	16,000	160,000
Pipeline	33,690,000	61,000	2,967,000
Pumps	7,641,000	1,461,000	2,120,000
Treatment	17,640,000	1,684,000	3,206,000
TOTAL			12,803,000

(2) Costs amortized over a 100-year period at 8-5/8 percent (Factor 0.08627)

Impacts on other project purposes. The Denison Dam (Lake Texoma) project was authorized by Public Law 75-761 (approved June 28, 1938) for flood control and other purposes as described in House Document Numbered 541, Seventy-fifth Congress, third session, with such modifications as deemed advisable by the Secretary of War and the Chief of Engineers. Public Law 76-968 (approved October 17, 1940) declared the project to be for the purpose of improving navigation, regulating flows of the Red River, controlling floods, and for other beneficial uses. Subsequent Public Laws (PL 83-273, approved August 14, 1953 and PL 85-146, approved August 14, 1957) authorized the Corps of Engineers to contract with the cities of Denison and Sherman, Texas, for water supply storage. Though not specifically designated as project purposes, under authority of other public laws and executive order, Lake Texoma provides lands and facilities for public recreation and the preservation and conservation of fish and wildlife resources.

The effects of the proposed reallocation on the project purposes, for which Lake Texoma was authorized, surveyed, planned, constructed, and operated (excluding hydropower generation) are outlined below.

Purpose	Impact	Discussion
Flood Control	None	Storage reallocation occurs in the conservation pool and does not effect flood control operations.
Improving Navigation	None	The project has never been operated for navigation nor has storage been assigned in the lake for this purpose. Downstream of Shreveport, Louisiana, the Red River Navigation System is under construction by the Lower Mississippi Valley Division. The projected minimum flow

		at Fulton, Arkansas, required to sustain navigation below Shreveport would be 1,056 cubic feet per second (cfs). The minimum recorded flow at this station, since impoundment of Lake Texoma, was 390 cfs in October 1956. In recent years, several reservoir projects have been completed in Tulsa District which raise low flows at Fulton. Broken Bow Lake with 470,100 acre-feet of storage for power, water quality, and water supply has had a significant impact. Hugo Lake releases flows for water quality which also increase the flows during dry periods. A synthesis of the operation of this system of reservoirs has been made using the latest information on power schedules. Although short-term flows could change due to power schedule changes, the lowest daily synthesized flow at Fulton was over 1,000 cfs. This was statistically shown to have a recurrence interval of about 50 years. If the assumptions in the synthesis of operation from 1938 through 1976 hold true, adequate flows for navigation should occur although no releases are specifically made for that purpose. Storage to sustain navigation below Shreveport is apparently not required in Lake Texoma. The synthesis of operation shows that at Shreveport, a flow of 1,056 cfs is exceeded 100% of the time, 2,000 cfs is exceeded 99% of the time and 3,000 cfs is exceeded 95% of the time.
Regulating Flow of the Red River	None	No scheduled releases are made for minimum downstream flow requirements. The proposed reallocation could change average annual and critical year average flows from 4,400 to 4,300 cfs and 1,350 to 1,250 cfs, respectively.
Other Beneficial Uses		
Water Supply	Beneficial	Increased storage allocated to water supply by 77,400 acre-feet.
Fish & Wildlife	None	No noticeable changes in lake levels or pool fluctuations. Pool may be slightly more stable.
Public Recreation	None	No change in recreation attendance or on public recreation lands or facilities.

<u>Impact of reallocation on the opportunity to add generating units at Denison Dam</u>. A preliminary assessment indicated that the addition of two 35 MW hydropower units at Denison Dam would be economically feasible with the existing water supply allocation of 72,600 acre-feet of storage and with the proposed reallocation of an additional 77,400 acre-feet of hydropower operated as one Federal hydropower generating station, the benefit-to-cost ratio of adding two 35 MW units would be about 3.9 to 1.0 without the reallocation and 3.5 to 1.0 with the proposed reallocation. The proposed reallocation would result in a net decrease in average annual energy output of about 200 MWH in 1985 increasing to 600 MWH by 2044 due to the effect of anticipated sediment accumulation. This would result in an average annual energy value benefit reduction of $23,000 without fuel escalation, and $58,000 with real fuel cost escalation. The

estimated present value of the loss would range from $275,000 to $685,000 depending upon the fuel escalation assumption used. The preliminary assessment was based on the following assumptions: (1) Thermal alternative is a coal-fired plant; (2) Federal construction and marketing; (3) Economic factors: October 1983 prices, Federal discount rate - 8-5/8%, and analysis period is 1990, to 2044; and (4) Generalized power values furnished by FERC 25 May 1984: project-on-the-line date - 1990, EIA fuels without escalation, and EIA fuels with escalation.

Conclusions

The reallocation of additional storage space to water supply does reduce the hydropower production capability. For this reduction, the SWPA is credited with the estimated benefits foregone from existing generating units at Denison Dam that result from a reallocation of 75,000 acre-feet of hydropower storage (the storage amount contracted to water supply). The credit began upon receipt of the first payment under the terms of the water supply contract and will be limited to the term of the current contract between SWPA and the Texoma power customers, which expires on December 31, 2003. The credit will be increased at a rate of 5% per year. Following the expiration or cancellation of the power sales contract, credits to SWPA will be reduced to revenues foregone. These annual revenue foregone charges will be shown as a direct recoverable charge against the water supply function up to the amount of revenues that would be foregone due to the reallocation and credited to that account and effect an equivalent reduction in OM&R charges to the existing hydropower purpose of the project.

The credits beyond the traditional revenues foregone are intended to allow SWPA to provide compensation to the Electric Cooperatives for lost hydropower generation at Denison Dam. Based upon discussions conducted among representatives of the Electric Cooperatives, the North Texas Municipal Water District, the SWPA, and the Department of the Army on May 2, 1985, it was understood that the contract between SWPA and the Cooperatives would be amended to reduce payments by the Cooperatives and thereby provide compensation for the loss of generation. The specific amount of the reduction in payment was negotiated by the SWPA and the Cooperatives. The approach to providing compensation to parties adversely impacted by storage reallocations described herein was not to be interpreted as establishing new policy for storage reallocations. The decision to allow compensation was based on the unique circumstances of the specific situation under consideration, including the electrical isolation of the cooperatives and the fact that 100% of the impacts of reduced energy production resulting from the reallocation fell on only two parties, the cumulative size of the storage reallocations at the Lake Texoma Project, the nature of the power service contract between the Cooperatives and the SWPA, and the nature of the authorizing legislation for Denison Dam.

Shortly after the 1985 reallocation by the Assistant Secretary of the Army (Civil Works), interests outside the Government began efforts to legislate authority for the Secretary to reallocate up to an additional 300,000 acre-feet of hydropower storage to water supply in Lake Texoma. Presumably, these interests wanted Congressional authorization for future water supply should the need arise. Congressional assurance of reallocation authority for the Secretary would prevent many of the challenges encountered in the North Texas Municipal Water District contract. As water supply interests pressed for assurances of storage availability, the electric co-ops pressed for guarantees of financial compensation if such reallocations occurred.

In spite of the Secretary's language concerning the credits for thermal replacement not setting a precedent, the electric customers succeeded in obtaining that legislative guarantee in the Water Resources Development Act of 1986 (PL99-662). This law also granted authority for the Secretary to reallocate up to 150,000 acre-feet of storage to water supply for entities in Texas and 150,000 acre-feet to entities in Oklahoma in addition to the existing 150,000 acre-feet reallocation.

On 28 February 1990, the North Texas Municipal Water District made an early payoff of \$15,932,322.39 which was the remaining principal on its 75,000 acre-feet water supply storage contract.

On 16 Mar 1990, immediately following the North Texas debt payment, Southwestern Power Administration (SWPA) invoiced the Corps for \$8,102,231.13. That sum is the present worth of the annual escalating credits through the year 2044 (end of 100-year project life) even though the power contract with the existing co-ops terminates in 2003. Tulsa District has forwarded the SWPA request for lump sum credit to higher authority with the recommendation that no credits beyond revenues foregone be allowed after 2003. This action by SWPA indicates that balanced Federal books for project purposes is no longer the objective.

DETERMINING DEPENDABLE CAPACITY LOSSES FOR WATER SUPPLY REALLOCATION STUDIES

by

D. James Fodrea[1] and Richard L. Mittelstadt[2]

Introduction

Many of the recent storage reallocations at Corps of Engineers reservoirs for water supply involve reductions in the output of hydroelectric plants. If the discharge through a powerplant is reduced due to an upstream withdrawal for water supply, there will be a reduction in the plant's energy output. In many cases, there is also a reduction in the plant's dependable capacity, due either to a reduction in generating head, a reduction in the energy available to support the plant's capacity in low flow periods, or both.

This paper addresses the alternative methods for computing dependable capacity, with particular emphasis on a relatively new method, which is particularly suitable for computing dependable capacity losses at projects located in areas where hydropower represents only a small portion of the area's power generating resources. This method is appropriate for use in water supply reallocation studies in most parts of the United States.

Traditional techniques for estimating the dependable capacity of a hydropower project are based on worst-case scenarios. As a result, they usually underestimate the amount of capacity that can be provided with some degree of reliability. This is especially true for hydro projects that are part of power systems where steam plants generate most of the power.

The U.S. Army Corps of Engineers and the Federal Energy Regulatory Commission (FERC) have developed a method that gives a more realistic estimate of the dependable capacity of hydro plants in thermal-based power systems. The method is especially useful because most hydro plants in the U.S. operate in systems dominated by steam generation.

The amount of power, or capacity, that a hydro plant can deliver varies with time, because both streamflow and head vary with time. For example, at seasonal storage projects, capacity depends on the reservoir level, which defines head. When the reservoir is full, the power plant's full peaking capability will be available. If the pool has been drawn down, the plant's capacity will be reduced. At pondage projects, which have enough storage to permit daily peaking, the amount of capacity that is usable depends on how much streamflow is available to shape the releases to meet the daily peak power demand. At pure run-of-river plants, capacity is a direct function of the streamflow coming down the river at a given time. If the flows are low, the plant's capacity output will be low. If flows are high, the plant's capacity will be high.

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To quantify these variabilities, the power industry uses the term "dependable capacity." Dependable capacity refers to the amount of capacity that a facility can deliver with some degree of reliability (5).

Using Dependable Capacity

Why is it necessary to compute a hydro plant's dependable capacity? There are several reasons. Power system planners use dependable capacity to measure a hydro project's contribution to the system's peak load-carrying capability. Power marketers need to know how much dependable capacity a plant can provide when they are negotiating hydropower sales contracts. And, dependable capacity is required for analyzing the economic feasibility of a hydro plant.

This article focuses on the last application, economic analysis. Project planners, as well as many investors, test the economic feasibility of a hydropower installation by determining if the cost of the hydro plant is less than the cost of the thermal power plant that would most likely be built if the hydro plant were not. The thermal plant could be coal-fired steam, combustion turbine, or some other type.

In this type of analysis, the thermal plant construction and operating costs which are saved by building the hydro plant are considered the "benefits" of the hydro plant. Benefits fall into two categories. The capacity benefit represents the investment costs which have been saved. The energy benefit represents savings in operating costs.

Zeroing in on the first category, the capacity benefit is the cost of constructing enough thermal capacity to do the same job as the proposed hydro facility. To compute this benefit, the engineer has to do two things. First, he must identify the most likely alternative thermal plant. Second, he must determine how much thermal capacity is equivalent to the hydro plant's capacity. He does this by comparing the performance of the two plants in helping the power system meet peak loads.

To compare the performance of a hydro plant versus a thermal plant, the engineer must consider three factors. One is the fact that the forced outage rates of the two plants are different. Another is the flexibility advantage that a hydro plant usually has compared to a thermal plant. The third factor is the effect of hydrologic variations, which causes the capacity of the hydro plant to vary with time. This is compared to the peak output of the thermal plant which remains essentially constant.

With these three factors in mind, the U.S. Water Resources Council's Task Force on Water and Energy developed an equation to estimate thermal capacity that would be equivalent to a given hydro plant's capacity (6).

$$\text{Equivalent Thermal Capacity} = (DC) \times (HMA/TMA) \times (1+F)$$

where:
- DC = hydro plant dependable capacity in kW
- HMA = hydro plant mechanical availability in percent
- TMA = thermal plant mechanical availability in percent
- F = hydro plant flexibility adjustment

The comparison of the plant's forced outage rates is handled by dividing the hydro plant's mechanical availability (HMA) by the thermal plant's mechanical availability (TMA). A power plant's mechanical availability is the difference between perfect availability (100 percent) and the equivalent forced outage rate. Forced outage data is available from an annual report prepared by the North American Electric Reliability Council (3).

The flexibility advantage of a hydro project is accounted for by the (1+F) factor in the equation. A value of 5 percent is typically used for a project with no major operating restrictions. For further data, consult references (3) and (4) at the end of this paper.

The third and most important factor is the effect of hydrologic variations on power output. Dependable capacity (DC) is the term in the equation that accounts for this factor. Dependable capacity can be calculated in several different ways.

Methods for Determining Dependable Capacity

In the United States, three methods traditionally have been used to measure dependable capacity (4). They are:

- the critical month method
- the firm energy method
- the specified availability method

A fourth method, the average availability method, was developed in the early 1980's and has been successfully used to measure dependable capacity in areas where most of the power comes from thermal power plants (4).

Each method was developed to meet a particular set of circumstances, and each has advantages. In this paper, we will try to show that the average availability method is the best method for evaluating many hydro projects. However, none of the methods is universally applicable. One must choose the appropriate method for the particular project and power system being analyzed.

Critical Month Method

In a power system where hydropower is the dominant resource, such as systems in the Pacific Northwest, a conservative approach must be taken to define dependable capacity. A drought could affect the output of most of the region's generating resources. In such a system, a hydro plant's dependable capacity would be based on output under adverse load and streamflow conditions. Typically, the dependable capacity would be the plant's capability in a high demand month near the end of the reservoir critical drawdown period. The impact of hydrologic variations would be greatest at storage projects, where the power plant loses generating capability when the reservoir is drawn down.

Using the most adverse month can be overly conservative, however, if the streamflow period it's based on has an extremely low probability of recurrence. For example, studies show that the most adverse streamflow period in the Pacific Northwest is the 1928-32 critical period, which has a recurrence interval of well over 100 years. Using this period as the basis for dependable capacity would not make sense, because it is more conservative than the reliability criteria for the overall power system. So, the regional power system decided to base dependable capacity on the 1936-37 low flow period, which has a recurrence interval more consistent with the power system's reliability criteria.

At Libby, a typical storage project in this system, the reservoir would have been drawn down in January 1937 to a level where head would permit a maximum output of about 512 MW. Thus, the dependable capacity of Libby is limited to 512 MW, compared to a peaking capability of 604 MW at full pool.

The critical month method was developed in an era when hydropower provided an important share of the generation for many utility systems. It is still a valid method for measuring dependable capacity in a hydro-based power system. However, it is not appropriate to apply to hydro plants in thermal-based power systems, because it does not account for the diversity of non-hydro energy sources in such systems.

Firm Energy Method

In the southern and southwestern regions of the U.S., hydropower represents only a small part of overall power generation. Many hydro plants in these areas have been designed as low plant factor peaking plants, which have high generating capabilities compared to streamflow. At these projects, dependable capacity is usually constrained by the availability of energy during drought periods. Without water, the plants can't generate as much power.

In these systems, dependable capacity has traditionally been based on a firm energy requirement. For example, the criteria applied in the 1970's by the Southwestern Power Administration to Corps of Engineers' hydro projects in that region was that 1,200 kWh of firm energy had to be provided each year to make a kilowatt of capacity dependable. Firm energy is the energy available in the most adverse water year (or sequence of water years).

The output of hydro projects in these regions is now marketed on a system basis, so it is no longer possible to establish a generic requirement that applies to all projects. Each project is examined based on its contribution to the system's dependable capacity output. However, the firm energy method can be used to define the dependable capacity of the entire system. Marketing agencies use this general approach when they evaluate the marketable capacity of a new hydro project.

The firm energy method was developed to evaluate the dependable capacity of hydro in a combined hydro-thermal system, but it uses criteria that are nearly as conservative as the critical month method. This method may have been appropriate when the cost of constructing new generating facilities was relatively low and when system reliability was measured simply by a fixed reserve margin, such as 20 percent. But with today's high cost of construction, the power industry cannot afford such conservative criteria.

The firm energy method may still be useful in negotiating certain types of hydropower sales contracts. However, it should no longer be used as the basis of dependable capacity in economic feasibility studies because it often imposes a far stricter reliability requirement on hydro capacity than is applied to thermal plants in the same systems.

Specified Availability Method

The specified availability method is based on the assumption that, for the capacity of a small hydro project to be dependable, it must be available the same amount of time as the thermal alternative's capacity.

For example, assume the alternative to a given hydro plant is a coal-fired steam plant, which typically has a forced outage rate of about 15 percent. Available capacity would be determined by subtracting the forced outage rate and the maintenance outage rate from 100 percent capacity. However, since this method is usually based on conditions during the peak demand months when the thermal plants are not on maintenance, only the forced outage rate needs to be subtracted from total availability to arrive at the average availability. In this case, average availability of the

coal-fired steam plant would be 85 percent. Applying the same availability criteria to the hydro plant, its dependable capacity would be the capacity that's available 85 percent of the time.

To calculate dependable capacity, a capacity-duration curve would be constructed for the hydro plant. The 85 percent point on that curve defines the dependable capacity of the project, as shown in Figure 1.

This method was developed in the late 1970's, when interest was growing in small, run-of-river hydro projects. The critical month or firm energy methods could not be applied meaningfully to these types of projects. These methods assume that seasonal storage is available to regulate streamflows to meet demand, but most small hydro projects do not have seasonal storage.

The specified availability method attempts to relate hydropower reliability to thermal plant reliability. However, it ignores the hydro capacity that's available less than 85 percent of the time. When that capacity is available, it makes a contribution to meeting system peak power demand. But, the specified availability method gives the project no credit for this contribution. Consequently, this method usually underestimates a hydro plant's contribution to system load-carrying capability. Its value is limited to preliminary evaluations of small, run-of-river hydro plants.

"Dependable Capacity" of Thermal Plants

The three traditional methods are all based on worst-case or near worst-case criteria. If the dependable capacity of a thermal plant was based on the same philosophy, the plant might not have any dependable capacity. That's because, in the worst case scenario, the thermal plant would be completely out of service due to a forced outage.

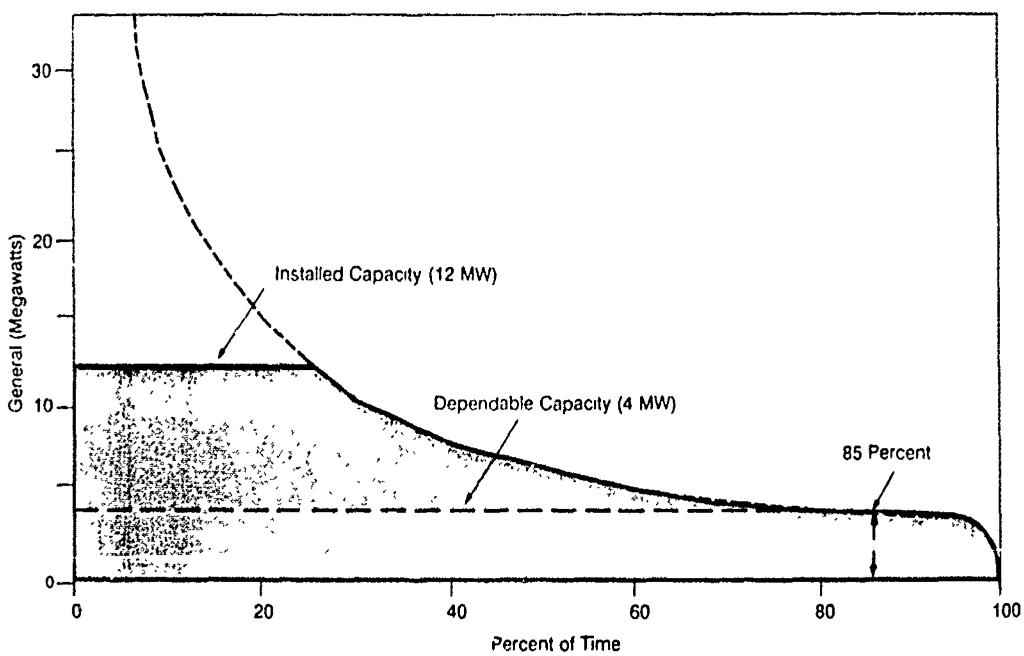

Figure 1: The shaded area under the duration curve shows the capacity distribution for a given 12 MW hydro plant. Using the specified availability method, this project would have 4 MW of dependable capacity.

How, then, do utilities measure the dependable capacity of a thermal plant? In reality, they wouldn't compute dependable capacity per se, but they would do something very similar. Standard practice is to determine how much additional peak load the plant will permit the system to carry. This value is measured with loss of load probability (LOLP) models that use probabilistic techniques to account for thermal plant forced outages.

If we are trying to estimate the dependable capacity of a hydro plant located in a diverse, thermal-based power system, it would seem logical to use the same as is method used for the thermal plants. In fact, such an approach would be mandatory if the object of the dependable capacity computation is to find the cost of the thermal alternative to the hydro plant. In other words, we should try to find out how much thermal capacity would be needed to make the same contribution to system peak load-carrying capability as the hydro plant. The average availability method was developed to measure hydro project dependable capacity in this way.

Average Availability Method

In the average availability method, system reliability is measured probabilistically. The availability of power at a hydro facility varies because of changes in head and/or streamflow. If the hydro plant is operated in a large, diverse power system, it should be possible to treat these variations in availability like the mechanical availability of thermal plants. In other words, the "derating" of a hydro plant because of reduced head or low streamflow is a statistical event similar to the derating or complete shutdown of a thermal plant due to a forced outage.

However, a problem arises when one tries to apply standard probabilistic techniques to hydro projects. A thermal plant's power availability can be shown in an LOLP model as "on," "off," or at one of several discrete levels of partial output. On the other hand, the hydro plant's available capacity is usually distributed continuously over a wide range of outputs, making it difficult to quantify.

In 1981, Gene Biggerstaff, chief of FERC's System Evaluation Branch, researched the problem of how to describe the hydrologic variability of a hydro project in such a way that it could be represented in a LOLP model (1, 6).

His basic source of data was a capacity-duration curve for a typical hydro project, similar to the project shown in Figure 2 which has an installed capacity of 178 MW. The curve represents the degree and amount of time the hydro project's installed capacity is derated due to reservoir drawdown, tailwater encroachment, and low streamflows. He broke the project's duration curve into horizontal segments, representing a series of "pseudo" power plants of various sizes and power availabilities. The dashed rectangles shown in Figure 2 depict the "pseudo" plants.

He then made a series of LOLP model runs to determine the amount of thermal capacity required to carry the same amount of system peak load as the hydro plant at the same level of reliability.

By applying this approach to various types of power systems, Biggerstaff found that it is not necessary to represent the availability of the hydro capacity as a series of power plants as long as the hydro project is relatively small compared to the size of the power system. Rather, it could be represented almost as accurately as a single "pseudo" plant having a capacity equal to the hydro plant's average capacity and an availability of 100 percent.

Computing Dependable Capacity Using Average Availability

To compute dependable capacity using the average availability method, one must have generation data covering a representative range of hydrologic conditions. This data can come either from historical operation data or from a long-term simulated operation study. To get a reliable results, at least 20 years of data is usually needed.

The analysis must be based on the time of year when power demand is greatest. For example, in the southern region of the U.S., the analysis would likely be based on data for the months of June through August. In areas where peak loads are about equal in summer and winter, both periods should be included in the analysis.

The next step is to convert the generation values and related data into a corresponding series of usable capacity values for the entire hydrologic period being analyzed. The dependable capacity is then calculated as the average usable capacity over the full range of historical head and streamflow conditions. Determining the usability of the capacity is a key part of this process.

In some analyses, the ratio of the usable capacity to the installed capacity is also computed. This ratio is called the hydrologic availability factor.

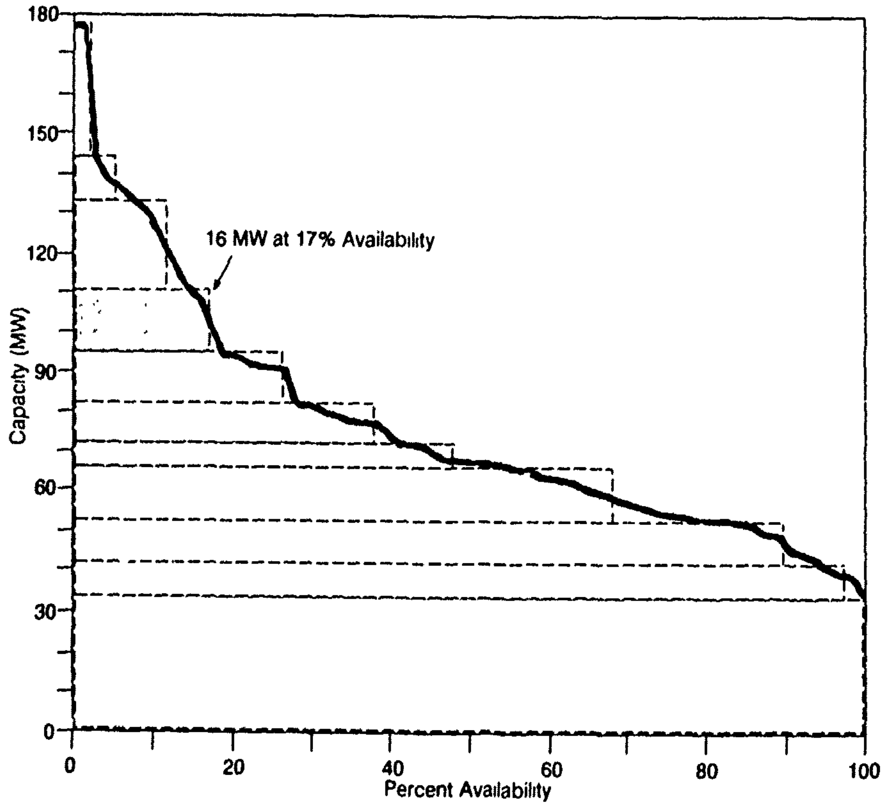

Figure 2: Capacity-duration curve for a typical hydro project with installed capacity of 178 MW. The ten dashed rectangles represent a series of "pseudo" plants that, together, represent the actual performance of a given hydro plant. The capacity of each "pseudo" plant is defined by the vertical dimension of the rectangle. The amount of time that the increment of capacity is available is defined by the horizontal dimension.

Usability of Capacity

A key point to remember is that dependable capacity must be based on the average capacity that is usable in the system load. Capaci‹; which has no water available to support it is not usable.

Hydro capacity can be "used" in s‹ ‹ ral ways: it can be run continuously to meet base load requirements, it can be operated several hours a day to meet intermediate or peak load requirements, it can be used to provide system reserve capacity, or it can serve a combination of these functions. The way the average usable capacity is calculated depends on the type of project and role which that project plays in meeting system loads.

For a pure run-of-river project, the analysis is relatively simple. The capacity that is available at any given time is a direct function of streamflow. The streamflow that is available on a given day to support the plant's capacity is often available all day, so that capacity is fully usable in the load. On some days, enough streamflow is available to use all of the plant's capacity. On other days, there may not be enough water to support all of the plant's capacity. The remaining, unsupported capacity has no use or value on those days. The average capacity for a run-of-river plant can be derived by converting a daily flow-duration curve for the peak demand months to a power-duration curve and integrating it to determine the average capacity.

At many hydro projects, there is either daily/weekly pondage or seasonal storage available to regulate streamflows to fit the daily and weekly load peaks. At projects like these, one must consider how the available capacity fits into the daily load shape.

The amount of time the capacity must be supported for it to be considered usable depends on the load characteristics of the individual system. A minimum requirement of four to six hours each weekday might be typical in some systems. In determining the amount of energy that is available for peak demand hours, analysts must allow for any minimum nighttime and/or weekend discharge requirements.

At some hydro projects, all or part of the plant's capacity is used to provide part of the system's reserve requirements. Reserves are necessary to pick up load temporarily in emergency situations, such as a forced outage to one of the system's primary generating plants or an unexpected load increase. A pumped storage project could be used in this way. Or, a conventional hydro project with some storage might have a portion of its capacity allocated to reserves during periods of low streamflow.

For hydro capacity to be usable as reserve capacity, it is not necessary to have enough energy to support that capacity day in and day out. However, enough water must be available to support it until a more permanent source of replacement power has been secured.

The amount of storage required for this service would depend on the characteristics of the power system in which the hydro project is being operated. Another requirement is that is must be possible to bring the hydro capacity on-line quickly in response to these emergency situations.

Usable capacity from each of the different types of operation discussed above is equally dependable, but each type of dependable capacity is not of equal dollar value. The value depends on what type of generating source would have to be built to provide that capacity if the hydro plant were not constructed. For example, if a hydro plant provided a given amount of dependable base load capacity, the thermal alternative would likely be coal-fired steam generation, which has a high investment cost. On the other hand, another hydro plant might supply an equal amount of dependable reserve capacity. Its dollar value would be based on the combustion turbine, which has a low investment cost per kW.

Application of the Average Availability Method

North Pacific Division, Corps of Engineers, has done several studies in the past few years using the average availability method for estimating the dependable capacity of hydro projects. The following example is based on a study that was made to determine how much dependable capacity would be lost when part of a project's storage is shifted from support of the hydro plant to providing municipal water supply. This project was located in South Atlantic Division.

The municipality proposed taking water directly from the reservoir, which would, in turn, reduce the amount of water released through the hydro plant. This results in a loss in energy production. Because this is a peaking project, the amount of energy available to support the capacity would be reduced. So, a loss in dependable capacity was considered likely.

Both the existing condition and the post-withdrawal condition were evaluated. The water supply withdrawal from the reservoir would reduce the energy output of the project by an average of 1,000 MWh per week.

The main source of hydrologic data was a system simulated operation study, which provided 63 years of weekly generation values for the project. This project is located in a summer-peaking power system, with a peak demand period running from mid-May to mid-September (weeks 20 through 37).

Table 1 shows capacity computations for several representative years and the average values for the 63-year period for both pre- and post-withdrawal conditions.

The project has no minimum discharge requirements, so all of the available energy can be used for peaking. However, the project is used for daily peaking, so for the capacity to be usable, energy must be available to support the capacity in each week of the peak demand period.

Power from the hydro plant is marketed as a part of a multi-project system. The marketable capacity for the system is based on the energy available in 1981, an adverse water year. In this water year, enough energy was available to support 330 MW of installed capacity for an average of 20 hours a week during the peak demand period. This criteria (20 hours per week) was used to determine how much capacity is usable in the load.

The average machine capability of the plant varies from year to year, depending on the average reservoir elevation during the peak demand season (machine capability represents the maximum capacity that can be supported for a given head condition). For each of the example years shown in the table, the average machine capability is greater than the 330 MW installed capacity. This is because the units have an overload capability that permits them to run at an output greater than the installed capacity when head permits.

Under existing conditions, sufficient energy is available with 1933 water to support the full 344 MW of machine capability for 32.9 hours per week. Under post-withdrawal conditions, the full 344 MW can still be supported, but for only 30.0 hours per week. However, this is still greater than the minimum requirement, so the facility experienced no loss in supportable, or dependable, capacity in this year. This type of situation exists in most years in the 63-year period of analysis.

However, with 1932 water, a somewhat different situation occurs. The 345 MW machine capability can be supported for 21.1 hours prior to the withdrawal, meeting the 20-hour minimum requirement. After the withdrawal, the 345 MW can be supported for only 18.2 hours. Therefore, average capacity that can be supported is limited to 314 MW (6,277 MWh/20.0 hours). In this year, the withdrawal causes a loss in supportable capacity of 31 MW.

TABLE 1

CAPACITY COMPUTATIONS SHOWING THE LOSS IN DEPENDABLE CAPACITY AT A HYDRO PROJECT WITH SEASONAL STORAGE AND 330 MW OF INSTALLED CAPACITY

	1932	1933	1981	1986	63-Year Average
Existing Condition					
Machine Capability (MW)	345	344	345	354	337
Average Energy (MWh)	7,277	11,324	6,600	5,844	12,785
Hours at Machine Capability	21.1	32.9	19.1	16.5	—
Required Peak Hours	20.0	20.0	20.0	20.0	—
Supportable Capacity (MW)	345	344	330	292	335
Post-Withdrawal Condition					
Machine Capability (MW)	345	344	344	351	336
Average Energy (MWh)	6,277	10,324	5,600	4,844	11,785
Hours at Machine Capability	18.2	30.0	16.3	13.8	—
Required Peak Hours	20.0	20.0	20.0	20.0	—
Supportable Capacity (MW)	314	344	280	242	331
Capacity Loss (MW)	31	0	50	50	4

Definition of Parameters

Machine Capability (MW):	Average machine capability of the plant over weeks 20-37, mid-May to mid-September.
Average Energy (MWh):	Average weekly generation over weeks 20-37.
Hours at Machine Capability:	Average number of hours the capacity can be supported per week at machine capability.
Required Peak Hours:	Hours per week that capacity must be available for it to be usable in the load.
Supportable Capacity (MW):	Capacity that can be supported by the average energy for the required 20.0 hours per week.
Avge. Supportable Cap. (MW):	Average capacity that can be supported over the entire 63-year period of record; this is the dependable capacity.

In 1986, there was not enough energy prior to withdrawal to support even the 330 MW installed capacity for 20.0 hours. Only 292 MW would be could be supported in that year. Post-withdrawal, the supportable capacity drops to 242 MW. The loss in capacity is 50 MW in 1986, the lowest streamflow year in the period of record.

Figure 3 uses a capacity-duration curve to summarize the capacity that can be supported in each of the 63 annual peak demand periods under both existing and post-withdrawal conditions.

The average capacity for the 63-year period under existing conditions is 335 MW. So, the dependable capacity of the project under the pre-withdrawal scenario is 335 MW. The average capacity under post-withdrawal conditions is 331 MW. Therefore, the proposed water supply withdrawal would cause a loss in dependable capacity of 4 MW.

Dependable Capacity vs. Marketable Capacity.

The average availability method for computing dependable capacity differs from the method used by some of the Power Marketing Administrations (PMA) in defining the amount of hydro capacity that they can market. For example, both the Southeastern Power Administration (SEPA) and Southwestern Power Administration (SWPA) use methods based on adverse water availability, because hydropower is their only generating resource. They can only guarantee delivery of the hydro capacity that they can support during adverse water conditions, because they have no thermal plants to back up the hydro. They sometimes purchase thermal power on the open market during dry periods, but doing so severely impacts their revenue rates and repayment obligations.

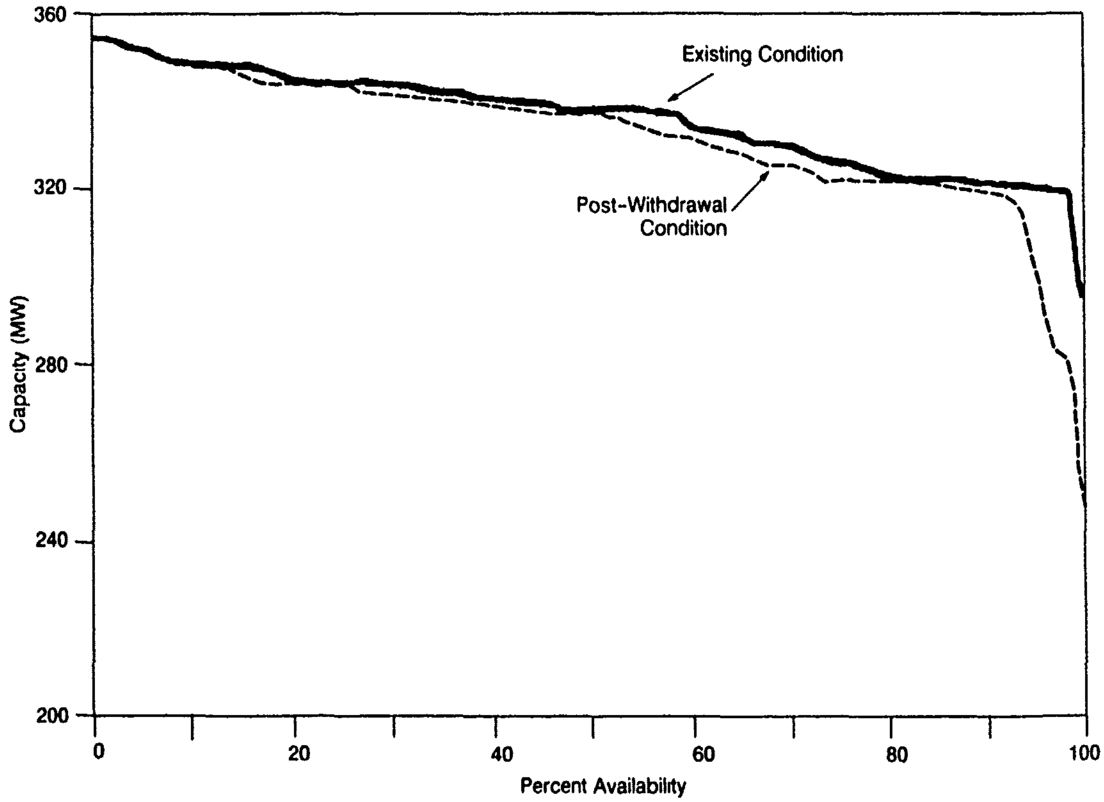

Figure 3: Capacity-duration curves showing distribution of supportable capacity at a hydro project over a period of 63 years under two different operating scenarios.

However, even though the PMA uses a method based on an adverse hydro year to determine the marketable capacity of its projects, that does not mean that their method is appropriate for measuring the loss in National Economic Development (NED) capacity benefits at these projects. The objective of NED benefits is to measure the gain or loss of benefits to the Nation as a whole[5], not to a single entity (such as SEPA) or to a small group of entities (SEPA's customers).

Referring to back to the example, the power from this project is marketed by the Southeastern Power Administration, and at the time the study was being made, SEPA was marketing capacity based on 1981 system capability, with 1981 being the second most adverse water year in the historical period of record. From Table 1 it can be seen that the capacity loss in that year was 50 MW. That value would be used in computing the capacity revenues foregone in the water supply reallocation study.

Conclusions

Traditional methods for computing dependable capacity are based on worst-case or near worst-case hydrologic conditions. These methods are appropriate for determining dependable capacity of plants in hydro-based power systems. But, they are too conservative in most parts of the country, where thermal plants are the dominant power source. The most accurate way to measure the dependable capacity of hydro facilities in such systems is to employ an approach similar to those used by utilities to evaluate thermal plant capacity, which is to measure the plant's contribution to system peak load-carrying capability.

The average availability method treats variations in hydro capacity availability due to changes in hydrologic factors like deratings and outages at thermal plants. It uses the basic principles of LOLP analysis, but in a manner that makes it much easier to account for the hydro plant's varying capacity output.

FERC has tested the method in a wide variety of power systems using a LOLP model and found that it gives reasonable results. The Corps of Engineers has successfully applied the average availability method in a number of recent hydro studies.

This method is recommended for estimating the dependable capacity of a hydro plant in a large, diverse thermal-based power system, which is typical of most power systems in the United States. Thus, it is suitable for evaluating the dependable capacity loss due to water supply reallocations at most Corps of Engineers projects. The only exceptions would be projects located in North Pacific Division, where the regional power system is hydro-based.

[5] It is not practical to examine the impact of a capacity loss on the entire national power grid, but NED benefits can be approximated by analyzing the regional power system, which is what was done in this study.

References

1. Biggerstaff, Gene, "Capacity Values and Adjustments," *Conference Proceedings, Waterpower '83: International Conference on Hydropower*, University of Tennessee, Department of Conferences and Non-Credit Programs, Knoxville TN, 1983, pp. 541-47.

2. Karadi, Gabor M. (ed), *Proceedings: International Symposium on the Dynamic Benefits of Energy Storage Plant Operation*, University of Wisconsin-Milwaukee, Milwaukee WI, 1984.

3. North American Electric Reliability Council, *Equipment Availability Report: 1979-88, North American Electric Reliability Council*, Princeton, N.J., issued annually.

4. U.S. Army Corps of Engineers, *Hydropower Engineering and Design* (EM 1110-2-1701), USACE, Washington D.C., 1985, Section 6-7.

5. U.S. Federal Power Commission, *Glossary of Important Power and Rate Terms*, USFPC, Washington D.C., 1965, page 2.

6. U.S. Water Resources Council, *Evaluating Hydropower Benefits*, USWRC, Washington D.C., 1981, Chapter 6.

REALLOCATION OF RESERVOIR STORAGE FOR WATER SUPPLY
ISSUES AND IMPACTS

by

Werner C. Loehlein[1]

INTRODUCTION

According to the Pennsylvania Department of Environmental Resources (PennDER), one of the most pressing water resource problems in the Commonwealth of Pennsylvania is that dealing with the consumptive use makeup requirements to protect the instream needs of the state's rivers and streams during low flow conditions. Through the State Water Plan, PennDER has identified the general magnitude of these consumptive water supply needs, and is in the process of determining feasible alternative solutions to these problems. Before PennDER pursues the development of new reservoirs, the Pittsburgh District of the U.S. Army Corps of Engineers, under Section 22 (P.L 93-251), was asked to examine the potential of its existing reservoirs for meeting all or a portion of theses needs. The potential of these reservoirs, by either reauthorization of storage or modification of the structure to add additional storage, was considered, with the initial effort by the Pittsburgh District directed to the Allegheny Reservoir in 1978. The Allegheny Reservoir study completed in 1980 was followed by studies of the Youghiogheny River Lake (1983), the East Branch Clarion River Lake (1984), and the Woodcock Creek Lake (1988). This paper will focus on two of these studies, i.e. Allegheny Reservoir where excess storage for water supply appears to be available, and Youghiogheny River Lake where no excess storage appears to be available.

ALLEGHENY RESERVOIR

<u>General</u>. The damsite is located on the Allegheny River in Warren County, Pa., approximately 198 miles above the mouth of the river at Pittsburgh, Pa. (Plate 1). The reservoir is located in Warren and McKean Counties, Pa., and Cattaraugus County, N.Y. The drainage area above the dam is 2,180 square miles. The project's purposes include: flood control, low-flow augmentation, hydropower, pollution abatement, water quality control, and recreation.

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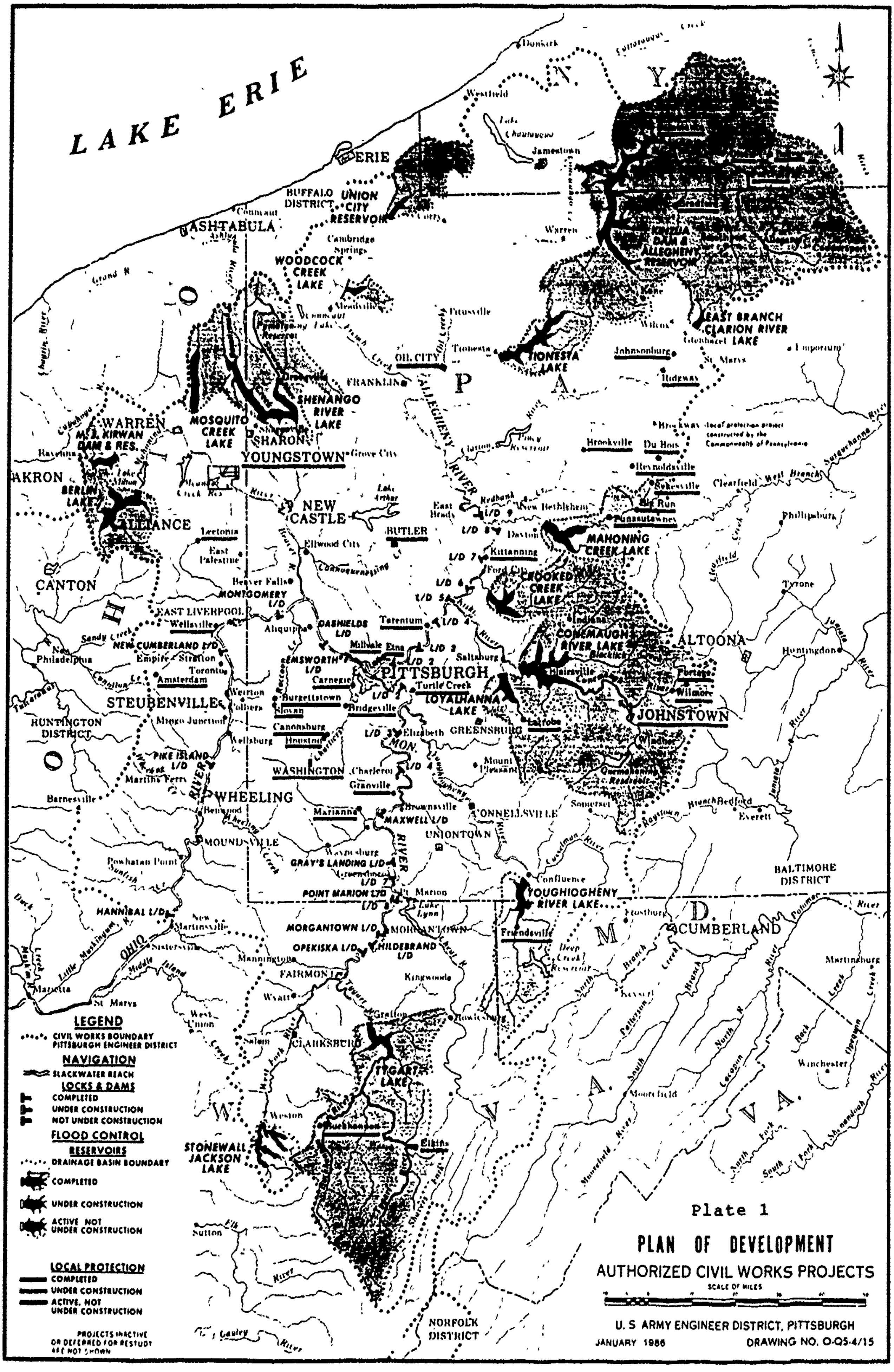
[1]Hydraulic Engineer, Pittsburgh District, U.S. Army Corps of Engineers


LAKE ERIE
Plate 1
PLAN OF DEVELOPMENT
AUTHORIZED CIVIL WORKS PROJECTS
SCALE OF MILES
U. S ARMY ENGINEER DISTRICT, PITTSBURGH
JANUARY 1986
DRAWING NO. O-Q5-4/15
LEGEND
CIVIL WORKS BOUNDARY PITTSBURGH ENGINEER DISTRICT
NAVIGATION
SLACKWATER REACH
LOCKS & DAMS
COMPLETED
UNDER CONSTRUCTION
NOT UNDER CONSTRUCTION
FLOOD CONTROL
RESERVOIRS
DRAINAGE BASIN BOUNDARY
COMPLETED
UNDER CONSTRUCTION
ACTIVE NOT UNDER CONSTRUCTION
LOCAL PROTECTION
COMPLETED
UNDER CONSTRUCTION
ACTIVE, NOT UNDER CONSTRUCTION
PROJECTS INACTIVE OR DEFERRED FOR RESTUDY ARE NOT SHOWN
BUFFALO DISTRICT
UNION CITY RESERVOIR
ERIE
ASHTABULA
WOODCOCK CREEK LAKE
KINZUA DAM & ALLEGHENY RESERVOIR
EAST BRANCH CLARION RIVER LAKE
TIONESTA LAKE
OIL CITY
SHENANGO RIVER LAKE
MOSQUITO CREEK LAKE
WARREN
M. J. KIRWAN DAM & RES.
SHARON
YOUNGSTOWN
AKRON
BERLIN LAKE
ALLIANCE
NEW CASTLE
BUTLER
MAHONING CREEK LAKE
CROOKED CREEK LAKE
CANTON
MONTGOMERY L/D
EAST LIVERPOOL
DASHIELDS L/D
CONEMAUGH RIVER LAKE
ALTOONA
NEW CUMBERLAND L/D
EMSWORTH L/D
PITTSBURGH
LOYALHANNA LAKE
STEUBENVILLE
JOHNSTOWN
HUNTINGTON DISTRICT
PIKE ISLAND L/D
WASHINGTON
GREENSBURG
WHEELING
MAXWELL L/D
CONNELLSVILLE
UNIONTOWN
GRAY'S LANDING L/D
BALTIMORE DISTRICT
POINT MARION L/D
YOUGHIOGHENY RIVER LAKE
HANNIBAL L/D
MORGANTOWN L/D
CUMBERLAND
OPEKISKA L/D
HILDEBRAND L/D
FAIRMONT
CLARKSBURG
TYGART LAKE
STONEWALL JACKSON LAKE
NORFOLK DISTRICT

Study Development. A daily flow simulation program for Allegheny Reservoir was developed by the Hydrology and Hydraulics Branch of the Pittsburgh District. The original version of the program, developed in 1967, was revised, updated and made executable on the Computer Sciences Corporation - Univac 1108 computer. Using the present storage allocation schedule (Plate 2), the pre-Kinzua Dam period (1929-66) was modeled.

Kiskiminetas River Effect. A water quality investigation of the effect of the Kiskiminetas River on the Allegheny River was made to determine the minimum amount of augmentation required from Allegheny Reservoir to maintain stream quality during the low-flow period. From June through September 1977, water samples were collected on six different occasions, from the Kiskiminetas River near its mouth and from the Allegheny River, upstream of the Kiskiminetas River, at Freeport. A water sample analysis was conducted in the Water Quality lab of the Hydrology and Hydraulics Branch. Sample Ph values ranged from 6.8 to 7.4 for the Allegheny River, and 3.3 to 5.8 for the Kiskiminetas River. On each sample occasion, 1 ml. of Allegheny River water at a time was added and mixed to 100 ml. of Kiskiminetas River water and a pH reading was taken. This process was repeated until the pH of the mixed water equalled the pH of the added Allegheny River water. The results of the six sample analysis showed that for a pH = 6.5, the ratio of Allegheny River water to Kiskiminetas River water ranged from 0.35 to 4.90. The approach, therefore, proved to be inadequate for determining a minimum acceptable mixing ratio.

Conclusions about water quality guidelines are complicated by the many variables involved. For example, the range of pH values for the Kiskiminetas River is wide. The pH of the Kiskiminetas River near its mouth generally ranges from 2.5 to 6.0. The Allegheny River at Freeport has a pH range of 6.0 to 8.0. The degree of natural mixing also varies with flow and on some occasions with water temperature. When the Kiskiminetas water temperature falls below that of the Allegheny, the acidic Kiskiminetas water slides under the warmer Allegheny River and very little mixing occurs. Also during low-flow conditions, the Kiskiminetas River, which enters the Allegheny from its left or east bank, remains along that bank, again with very little mixing.

During low Allegheny River flows, especially following a drought, the Pennsylvania Department of Health in a report (Kiskiminetas - Allegheny River Water Quality - dated 7 November 1966) stated that Kiskiminetas River flow should not exceed about 15 percent of the Allegheny River flow. This percentage was considered necessary to prevent fish kills in the Allegheny River. A more recent investigation of water quality and stream flow data since Allegheny Reservoir went into operation in 1966 indicated that this percentage can increase as Allegheny River flows increase

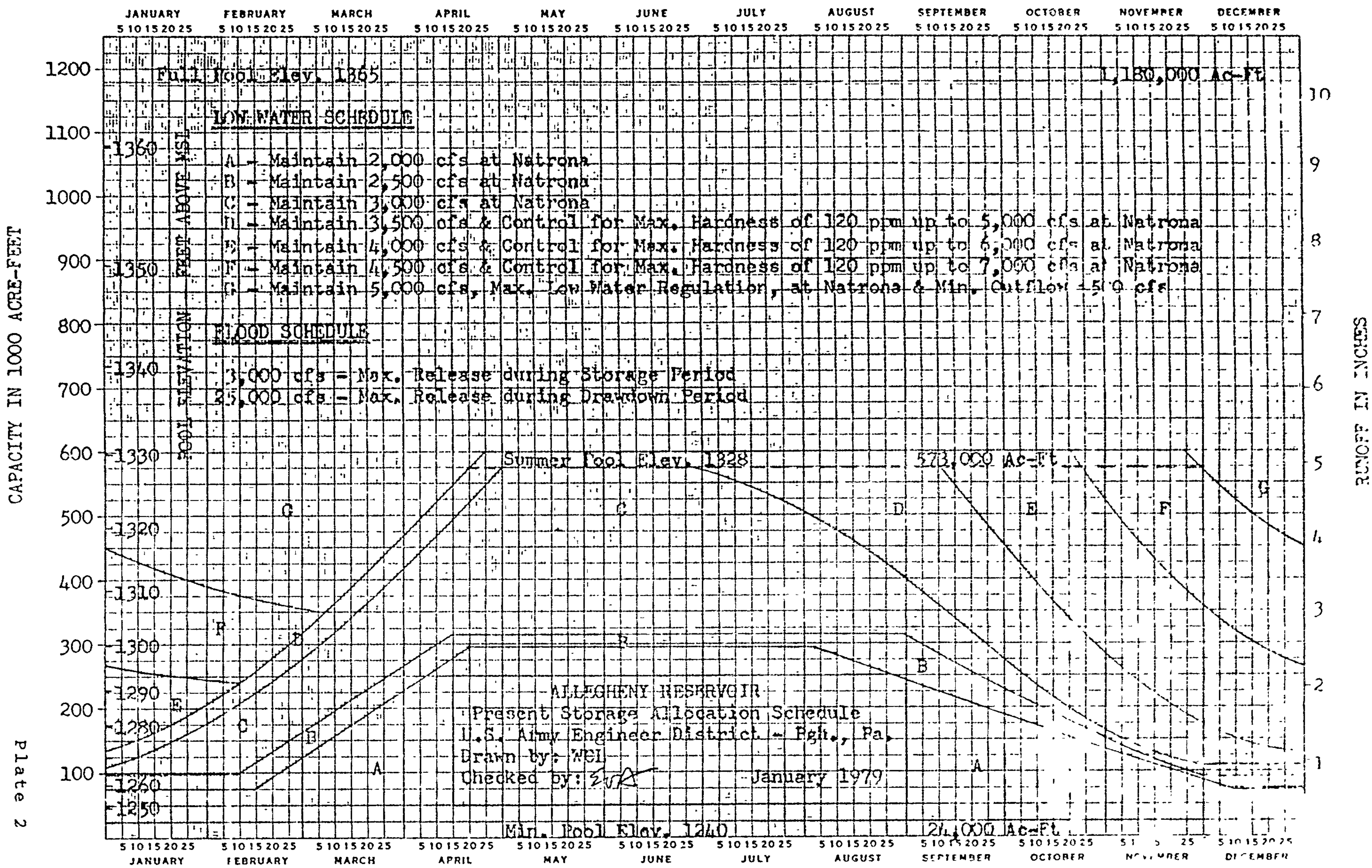

Full Pool Elev. 1365
1,180,000 Ac-Ft
LOW WATER SCHEDULE
A - Maintain 2,000 cfs at Natrona
B - Maintain 2,500 cfs at Natrona
C - Maintain 3,000 cfs at Natrona
D - Maintain 3,500 cfs & Control for Max. Hardness of 120 ppm up to 5,000 cfs at Natrona
E - Maintain 4,000 cfs & Control for Max. Hardness of 120 ppm up to 6,000 cfs at Natrona
F - Maintain 4,500 cfs & Control for Max. Hardness of 120 ppm up to 7,000 cfs at Natrona
G - Maintain 5,000 cfs, Max. Low Water Regulation, at Natrona & Min. Outflow 500 cfs
FLOOD SCHEDULE
3,000 cfs - Max. Release during Storage Period
25,000 cfs - Max. Release during Drawdown Period
Summer Pool Elev. 1328
573,000 Ac-Ft
ALLEGHENY RESERVOIR
Present Storage Allocation Schedule
U.S. Army Engineer District - Pgh., Pa.
Drawn by: WCI
Checked by:
January 1979
Min. Pool Elev. 1240
24,000 Ac-Ft
CAPACITY IN 1000 ACRE-FEET
POOL ELEVATION - FEET ABOVE MSL
RUNOFF IN INCHES
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

Plate 2

and still provide adequate water quality. For the period 1972-77, Allegheny River flows at Natrona, downstream of the Kiskiminetas River, were compared to the ratio of the Kiskiminetas flow at Vandergrift and the flow of the Allegheny at Natrona. Instances of water quality problems and days of duration were noted. Zones of danger, transition and adequate Allegheny River water quality were then determined. Plate 3 shows the Allegheny River water quality guidelines for minimizing the effects of the Kiskiminetas River that were developed.

<u>Potential Water Supply Determination</u>. The July 1930-April 1931 drought period was modeled to test the viability of providing the 7-day, once in 10-year low flow of 2,900 c.f.s. as regulated at Natrona. This period was chosen because it is the most severe of record and is estimated to have a recurrence frequency of greater than once in 100 years. The 7-day, once in 10-year low flow was chosen because it is a water quality standard on which treatment facility design is based and is identified in the State Water Plan. Minimum flows of 2,000 and 2,500 c.f.s. at Natrona were also tested. hese flow values are the scheduled flows at Natrona for Zones A and B, respectively, of the present storage allocation schedule. The Allegheny River water quality guidelines for minimizing the effects of the Kiskiminetas River were incorporated into the Allegheny Reservoir daily flow simulation program. While operating the reservoir to satisfy these newly developed Allegheny River water quality guidelines, and setting the storage allocation schedule at maximum conservation pool, computer runs maintaining minimum flows at Natrona of 2,000, 2,500, and 2,900 c.f.s. were made. The computed excess storage available amounted to approximately 316,000, 193,000, and 76,000 Ac-Ft, respectively. In each computer run, drawdown began on 4 July 1930; the lowest storage level occurred on 25 January 1931; recovery from the drought began on 13 February 1931; and summer pool was reached, as scheduled, on 20 April 1931. This indicated that the 7-day, once in 10-year low flow of 2,900 c.f.s. as regulated at Natrona could be maintained, adequate Allegheny River water quality would be provided, and up to 76,000 Ac-Ft of storage could be made available for water supply. At this point, maintained flows of 2,000 and 2,500 c.f.s. were dropped from further consideration because the results indicated that there would be a significant change in the 7-day, once in 10-year low flow at Natrona. Any significant change was considered undesirable.

To provide 76,000 Ac-Ft of storage as water supply, revisions to the existing storage allocation schedule were made. Primarily, Zones A and B were eliminated because they provided less than 2,900 c.f.s. at Natrona and assure adequate Allegheny River water quality. The drawdown portion of Zone D was adjusted to take better advantage of the wet season. Using this modified storage allocation schedule and utilizing the entire 76,000 Ac-Ft as block storage during the normally dry months of September and

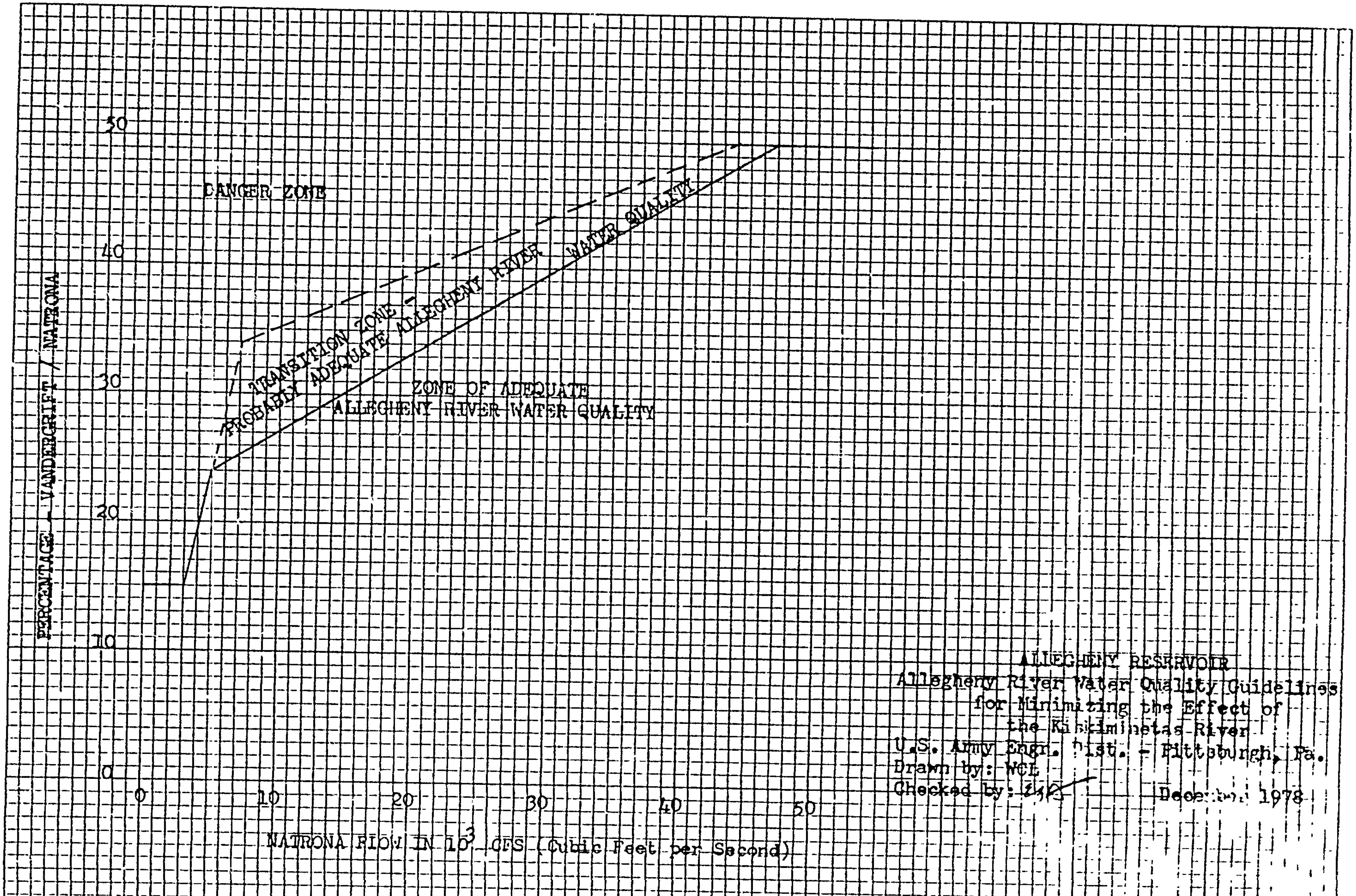
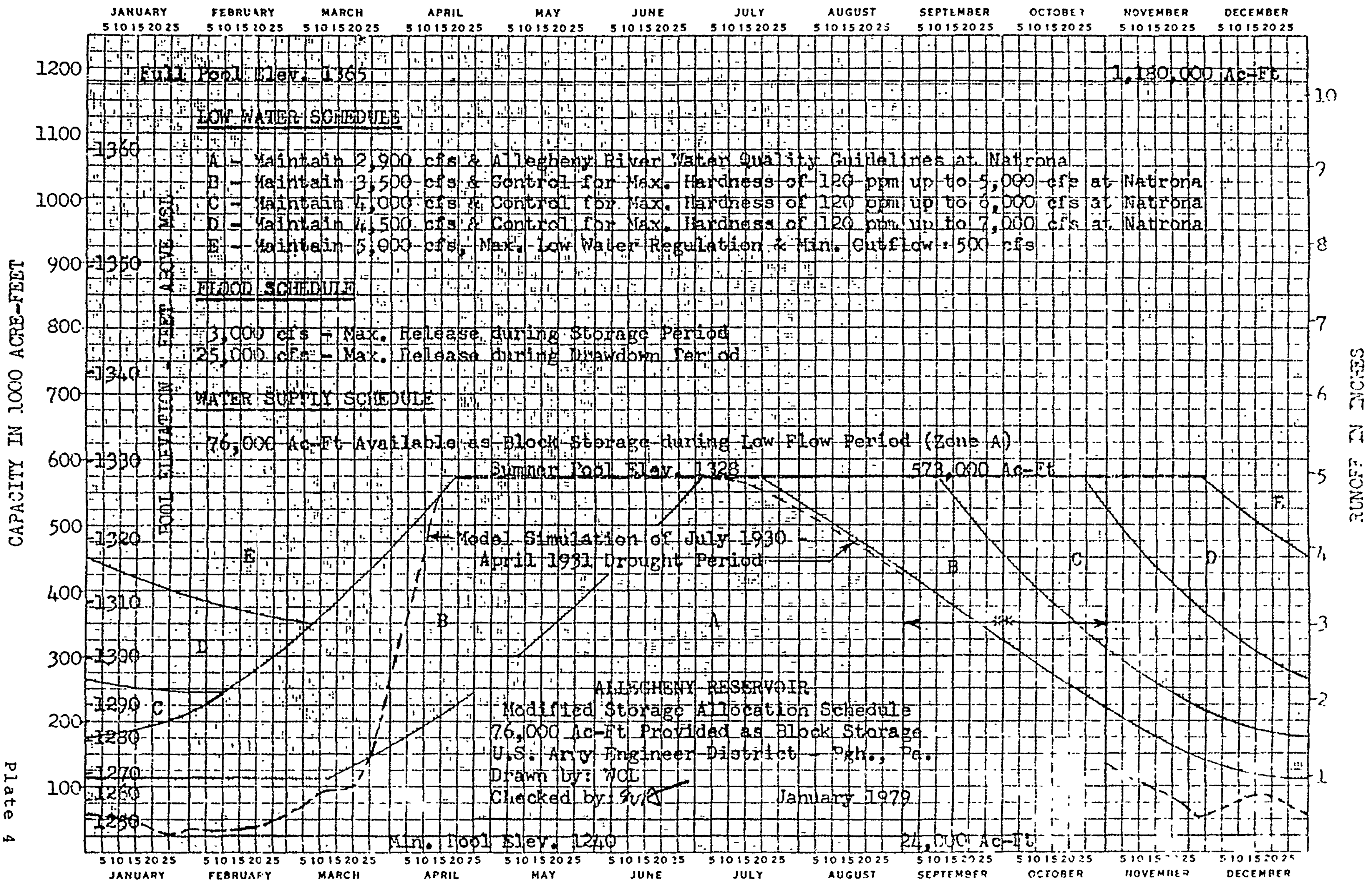

DANGER ZONE
TRANSITION ZONE
PROBABLY ADEQUATE ALLEGHENY RIVER WATER QUALITY
ZONE OF ADEQUATE
ALLEGHENY RIVER WATER QUALITY
PERCENTAGE - VANDERGRIFT / NATRONA
50
40
30
20
10
0
0
10
20
30
40
50
NATRONA FLOW IN 10^3 CFS (Cubic Feet per Second)
ALLEGHENY RESERVOIR
Allegheny River Water Quality Guidelines
for Minimizing the Effect of
the Kiskiminetas River
U.S. Army Engr. Dist. - Pittsburgh, Pa.
Drawn by: WCL
Checked by:
1978

Plate 3

October, the model simulation of the 1930-31 drought is shown in Plate 4. During the 1930-31 drought period a minor rise did occur in December. If no water supply was utilized as block storage before this rise a change in operating procedures would occur. This change would maintain a higher minimum flow downstream. Therefore, if the entire block storage was desired after mid-November and before mid-March, only 54,000 Ac-Ft could be made available to water supply. If water supply was desired at a constant daily rate, more total volume of storage (83,500 Ac-Ft versus 76,00 Ac-Ft) could be provided. During the 1930-31 drought, when final recovery from the drought began on 13 February 1931, there existed 83,500 Ac-Ft of storage above minimum pool. This storage could be equally distributed over the entire 1930-31 drought drawdown period, thus making 370 Ac-Ft per day (185 c.f.s.) of storage available to water supply. Further revision of the drawdown portion of Zone D would be required to incorporate the 1930-31 drought drawdown. It should be noted that current Corps of Engineers' policy is to sell a block of storage and not provide a constant rate as analyzed. However, future policies may change, therefore, both approaches were analyzed.

The majority of storage became available by maintaining 2,900 c.f.s. instead of 3,000 c.f.s. at Natrona, while drawdown was in Zone C of the present schedule. Also, the model adhered precisely to the operating procedures and only a limited margin of error was incorporated into the model. For example, when the storage allocation schedule required 2,900 c.f.s. at Natrona, the model provided exactly 2,900 c.f.s., whereas in real time operation this type of accuracy is very unlikely. Furthermore, since the distance from Kinzua Dam to Natrona is 174 miles and the time of travel during low flows ranges from 3 to 10 days, routing as well as the amount of a release from Allegheny Reservoir becomes significant. Routing in the model was simplified to be a specific increment of time for a range of flows at Natrona and adjusted to accommodate changes in travel times. Therefore, to provide additional flexibility in real time operation of the reservoir it would be desirable to utilize only a portion of the total available potential amount for other uses.

<u>Affects of Modification on Authorized Purposes</u>. Using the modified storage allocation schedule in Plate 4, a review of actual stream flow records for the years 1929 through 1966 was made to determine typical reservoir drawdown rates. The results of this review were compared to the drawdown curves of the present storage allocation schedule for the same period. The net effect of the change is to delay the rate of drawdown 3 to 6 days with a net increase of 1.2 feet in the pool level by the end of



** - Entire Water Supply Allocation Used during this Period

Plate 4

October for both a normal (50 percentile) and dry (75 percentile) year. This delay in drawdown causes no noticeable problems and is a minor benefit to recreation by prolonging the use of boat launches, beaches and other related activities.

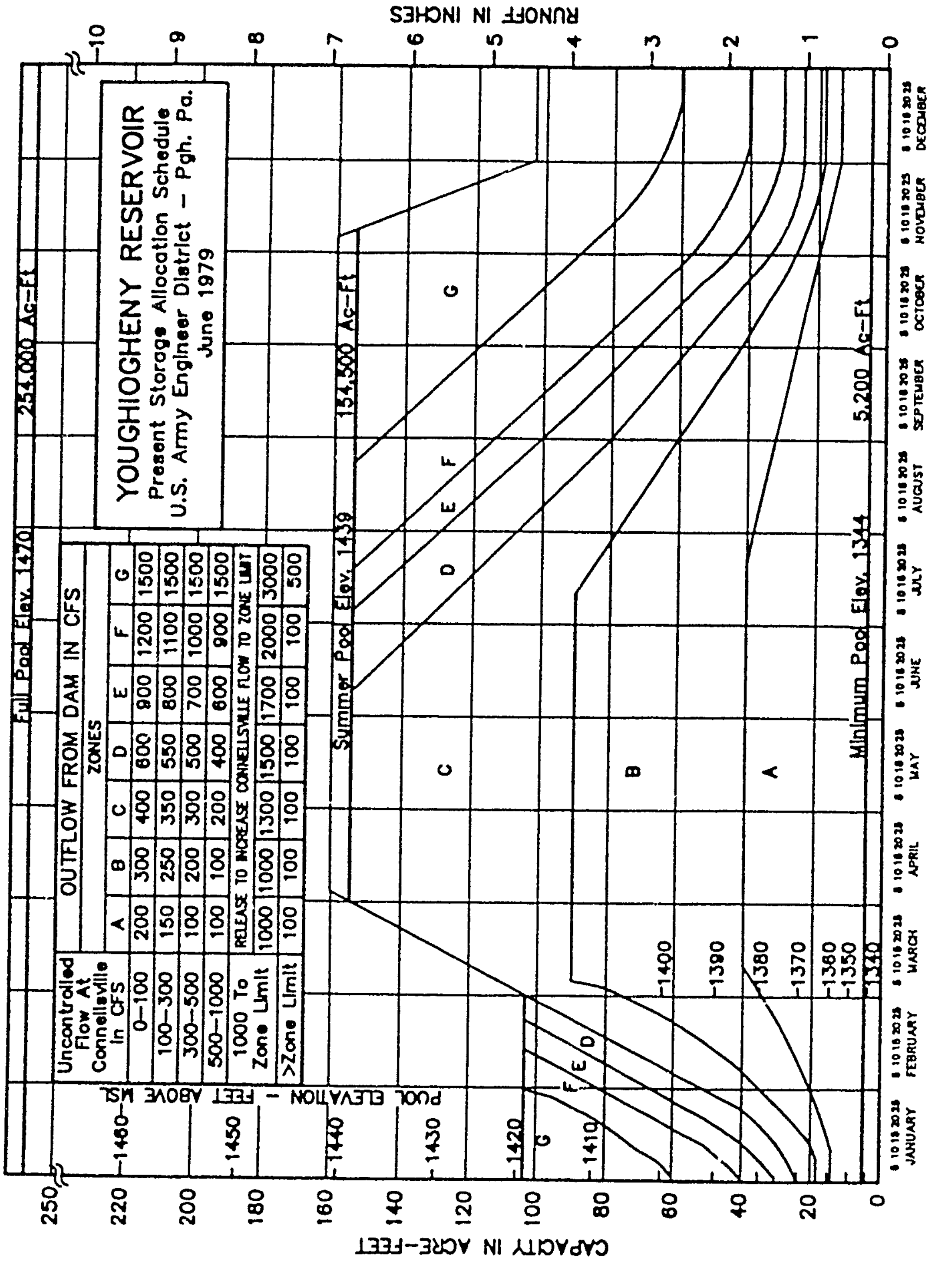
Allegheny Reservoir Summary and Conclusions. From the study a determination of the water supply potential of the Allegheny Reservoir shows that up to 83,500 Ac-Ft of storage could be made available to water supply if approved by Congress and the present storage allocation schedule revised. The study recommended that the computed storage be limited to only 45,000 Ac-Ft. This recommended volume was based on a study of project conditions, flexibility of reservoir operation, downstream water quality and current water quality criteria.

YOUGHIOGHENY RIVER LAKE

General. The dam is located on the Youghiogheny River about 74.2 miles above its junction with the Monongahela River at McKeesport, Pa., and 1.2 miles above Confluence, Pa. (Plate 1). The reservoir is located in Fayette and Somerset Counties, Pa., and Garrett County, Md. The drainage area above the dam is 434 square miles. The project's purposes include: flood control, low-flow augmentation, water quality control, and recreation.

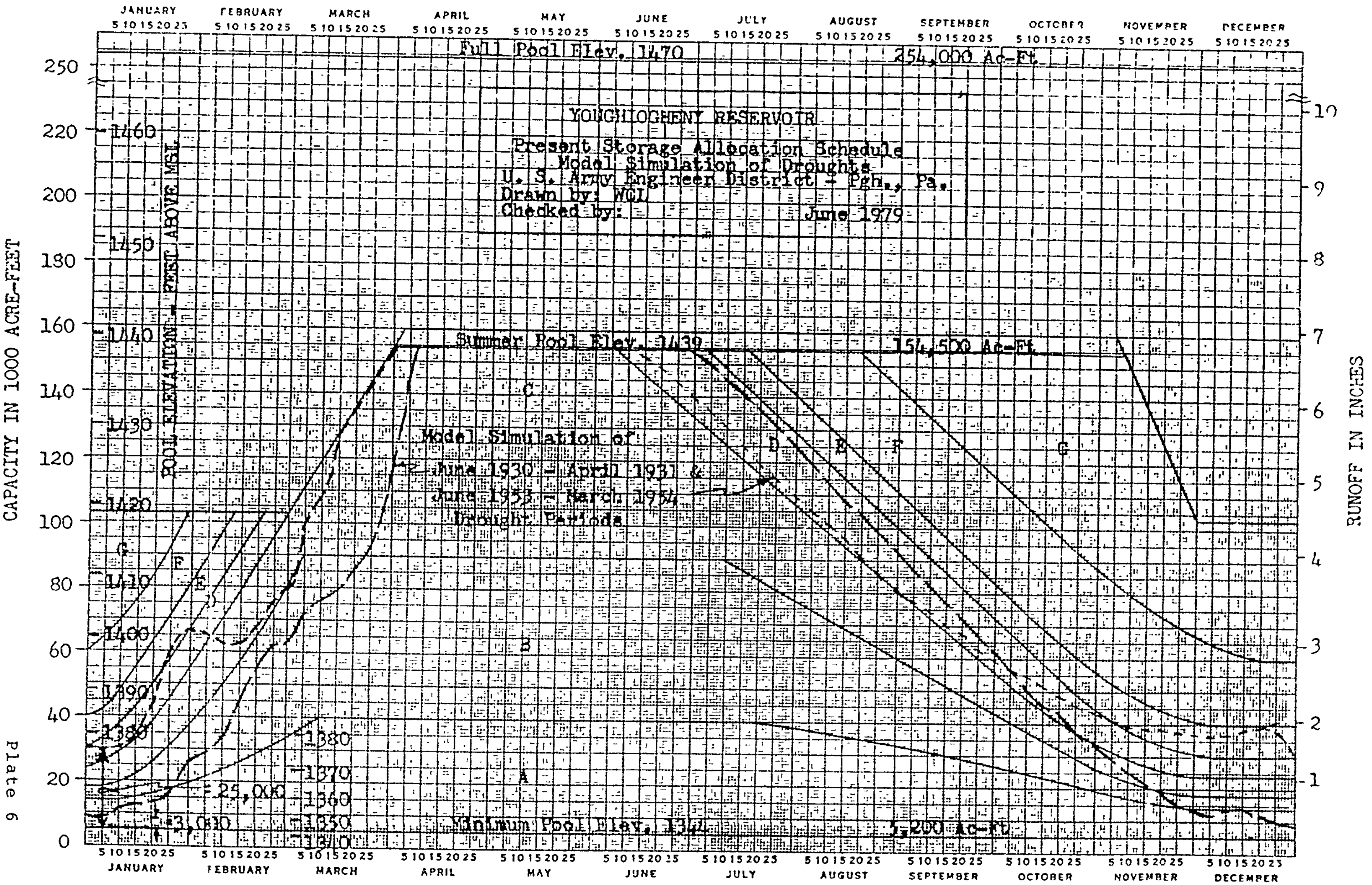
Study Development. A five-day average flow simulation program for Youghiogheny River Lake was developed by the Hydrology and Hydraulics Branch of the Pittsburgh District. The original version, developed in 1966, was revised, updated and made executable on the Boeing Computer Services (BCS) - CDC Cyber 175 Computer. All flow data was reviewed for consistency and adjusted where necessary. Using the present storage allocation schedule (Plate 5), the period 1929-77 was modeled. Special attention was paid to the June 1930-April 1931 and June 1953-April 1954 drought periods. The 1930-31 drought was the most severe of record and is estimated to have a recurrence interval of about 100 years. The 1953-54 drought was the most severe since Youghiogheny River Lake went into operation. Using the present storage allocation schedule, the model simulations computed a minimum five-day average excess storage of approximately 3,000 Ac-Ft for the 1930-31 drought and approximately 25,000 Ac-Ft for the 1953-54 drought. Plate 6 shows the model simulation of these two drought periods.

Alternative Water Quality Criteria. The Pittsburgh District's Youghiogheny River Lake Water Quality report, dated June 1978, stated that some substantial seasonal variations have been noted in the effectiveness of operations to mitigate acid mine drainage in the Youghiogheny River downstream of the project. Particularly, when low-flow conditions prevailed, which



Uncontrolled Flow At Connellsville In CFS	OUTFLOW FROM DAM IN CFS — ZONES A	B	C	D	E	F	G
0–100	200	300	400	600	900	1200	1500
100–300	150	250	350	550	800	1100	1500
300–500	100	200	300	500	700	1000	1500
500–1000	100	100	200	400	600	900	1500
	RELEASE TO INCREASE CONNELLSVILLE FLOW TO ZONE LIMIT						
1000 To Zone Limit	1000	1000	1300	1500	1700	2000	3000
>Zone Limit	100	100	100	100	100	100	500

Plate 5



YOUGHIOGHENY RESERVOIR
Present Storage Allocation Schedule
Model Simulation of Droughts
U. S. Army Engineer District - Pgh., Pa.
Drawn by: WGL
Checked by:
June 1979
Full Pool Elev. 1470
254,000 Ac-Ft
Summer Pool Elev. 1439
154,500 Ac-Ft
Minimum Pool Elev. 1344
5,200 Ac-Ft
Model Simulation of
June 1930 - April 1931 &
June 1953 - March 1954
Drought Periods
CAPACITY IN 1000 ACRE-FEET
POOL ELEVATION - FEET ABOVE MSL
RUNOFF IN INCHES
25,000
3,000
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

Plate 6

when maximum acid concentrations occur on the Casselman River, it has been noted the pH dropped for short periods of time in the Youghiogheny River. Because climatic and water quality conditions vary from year to year, it is impossible to estimate precisely what percentage (or balance) of flow from the Casselman River can produce a significant pH depression in the Youghiogheny River at Connellsville, Pa. However, during the period 1958-68, when the flow of the Casselman River at Markleton exceeded 35% of the total flow of the Youghiogheny River at Connellsville, pH at Connellsville at times were depressed to 5.0 or less. In more recent years, flow contributions of 35% or more from the Casselman River above Markleton depressed the pH at Connellsville to around 6.0. The possibility of maintaining a ratio of Casselman River flow less than or equal to 35% of the total flow at Connellsville was therefore analyzed.

Examination of the model simulations using the present operating procedures indicated that this situation occurs annually and release of water would be required to maintain this 35% ratio. Incorporating this feature into the model produced results that indicated Youghiogheny Lake would operate to maintain this ratio an average of 21 days per year. The model further indicated that for approximately two of every three years, this ratio could be maintained all year with little difficulty. However, during a prolonged dry period, Lake storage would become deficient. A computer model simulation of the June 1930 to April 1931 drought indicated the lake would be deficient by more than 15,000 Ac-Ft. For the most part, the greatest difficulty in maintaining the 35% ratio all year was in attaining summer pool as scheduled by 1 April. In those particular years, summer pool was attained an average of four weeks late and as late as early June. In the water quality report, it was indicated that providing this ratio is most beneficial during the filling portion of the storage and release schedule (Plate 5). The results of maintaining this ratio only during March and April again indicated that for approximately two of every three years, little difficulty would be experienced. Youghiogheny River Lake storage would not become deficient even during a prolonged dry period. However, there remained the difficulty of attaining summer pool as scheduled. In those years, summer pool was attained an average of three and one-half weeks late and as late as the third week of May.

Downstream Recreation. The Youghiogheny River is considered very popular for canoeing and rafting. According to the Canoeing Guide to Western Pennsylvania and Northern West Virginia, published by the Pittsburgh Council of the American Youth Hostels, the 11 mile reach from Confluence to Ohiopyle is favorable for canoeists when the Confluence gage reads between 1.9 and 3.5 feet. Also, the seven mile reach from Ohiopyle to Stewarton is favorable for rafters and advanced canoeists when the Confluence gage height reads between 1.8 and 2.5 feet. At present the Confluence gage reads below 1.9 feet 5 percent of the time, and 2 percent of the time below 1.8 feet during the April

through October canoeing season. A flow of approximately 550 c.f.s. is equivalent to a Confluence gage height of 1.85 feet.

The possibility of incorporating maintained flows at Confluence for rafting and canoeing was investigated. Computer runs were made maintaining several different flows at Confluence. The model simulation indicated that a flow of approximately 700 c.f.s. (G.H. = 2.05 feet) could be maintained between April and October without causing Youghiogheny River Lake to become deficient. However, for the winter period between November and February, the Lake would be operated in the lower portion (Zone A) of the present release schedule.

<u>Alternative Low Flow Requirements</u>. The viability of providing the 7-day, once in 10-year low-flow of 440 c.f.s. as regulated and maintaining a Markleton to Connellsville flow percentage for water quality of less than or equal to 35% at Connellsville, was investigated. Setting the storage allocation schedule at the maximum conservation level, the 1930-31 and 1953-54 drought periods were simulated. The computed excess storage available amounted to 30,000 acre-feet for the 1930-31 period and 47,000 Ac-Ft for the 1953-54 period. The model simulations are shown on Plate 7. However, the filling portion of each drought period took approximately one month longer to reach summer pool than using the present storage allocation schedule. Summer pool was reached as late as 8 May, approximately 38 days behind schedule. Also, the model simulation of the 1953-54 drought period indicates that the pool would be drawn down early and very sharply in late May due to the high acid discharge from the Casselman River. Both of these situations were considered not ideal.

The possibility of obtaining excess storage by increasing the summer pool level of elevation 1439 (154,500 Ac-Ft) was also investigated. The initial approach taken was to extend the storage allocation schedule guide curves above the present summer pool level, leaving that portion of the present schedule below summer pool unchanged. The results were that for a prolonged dry period the net excess storage would change very little from that computed using the present schedule. The increased summer pool levels investigated were elevations 1441 (160,000 Ac-Ft) and 1444 (170,000 Ac-Ft).

<u>Revision of Existing Schedules</u>. The viability of revising the existing storage allocation and release schedules as outlined on Plate 5 was evaluated. The basic approach taken was to redistribute the lower portion of the release schedule by raising the scheduled flows in Zone A & B and reducing those in Zones C & D. Since limited margin of error was incorporated into the model and the model adhered precisely to operating procedures, it was considered that if a modified schedule would compute an excess

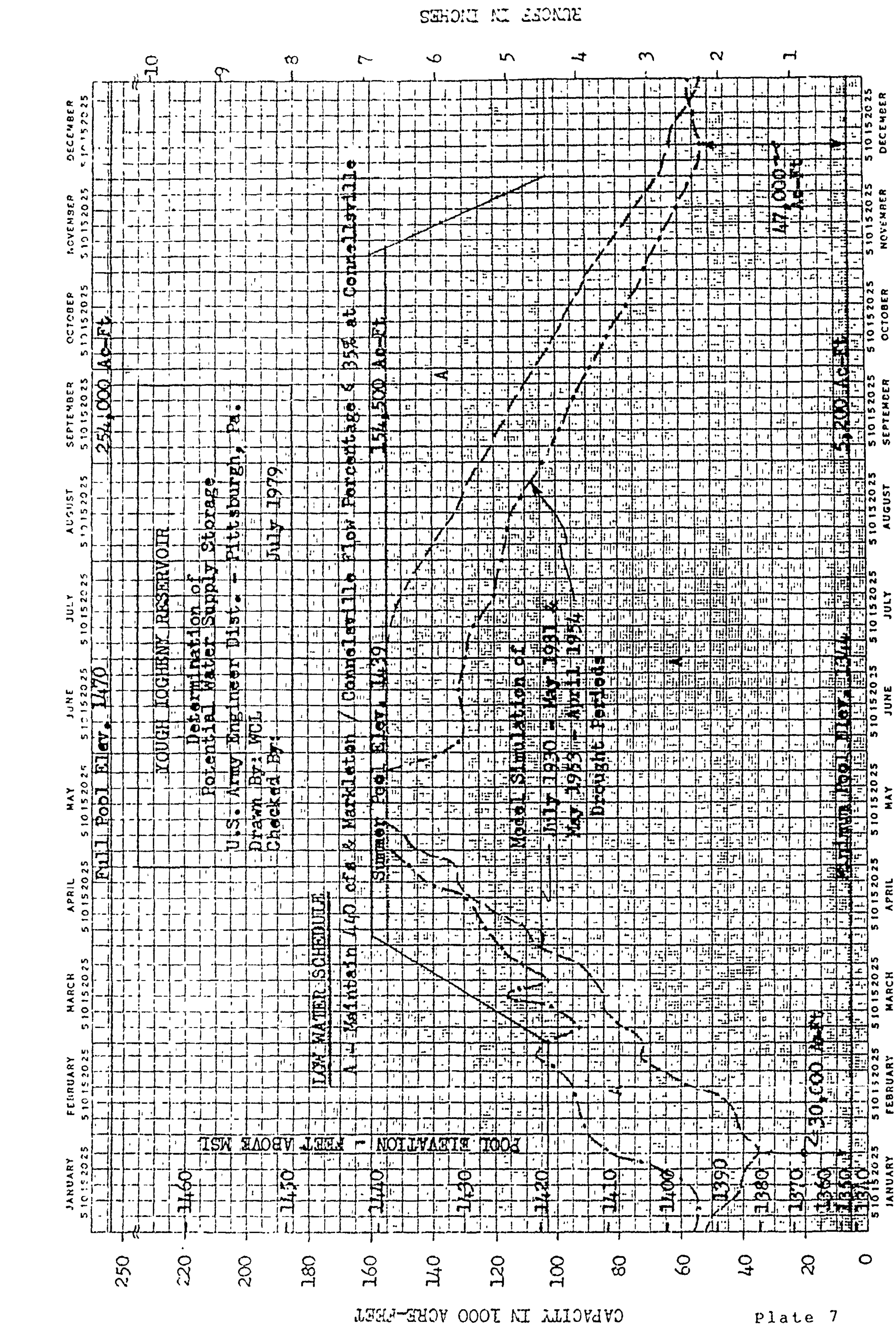

YOUGH LOGHENY RESERVOIR
Determination of
Potential Water Supply Storage
U.S. Army Engineer Dist. - Pittsburgh, Pa.
Drawn By: WCL
Checked By:
July 1979
LOW WATER SCHEDULE
A - Maintain 140 cfs @ Markleton / Connelsville Flow Percentage @ 35% at Connellsville
Full Pool Elev. 1470
254,000 Ac-Ft
Summer Pool Elev. 1439
154,500 Ac-Ft
Model Simulation of
July 1930 - May 1931 &
May 1953 - April 1954
Drought Periods
30,000 Ac-Ft
47,000 Ac-Ft
5,200 Ac-Ft
Minimum Pool Elev. 1344
POOL ELEVATION - FEET ABOVE MSL
CAPACITY IN 1000 ACRE-FEET
RUNOFF IN INCHES
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

Plate 7

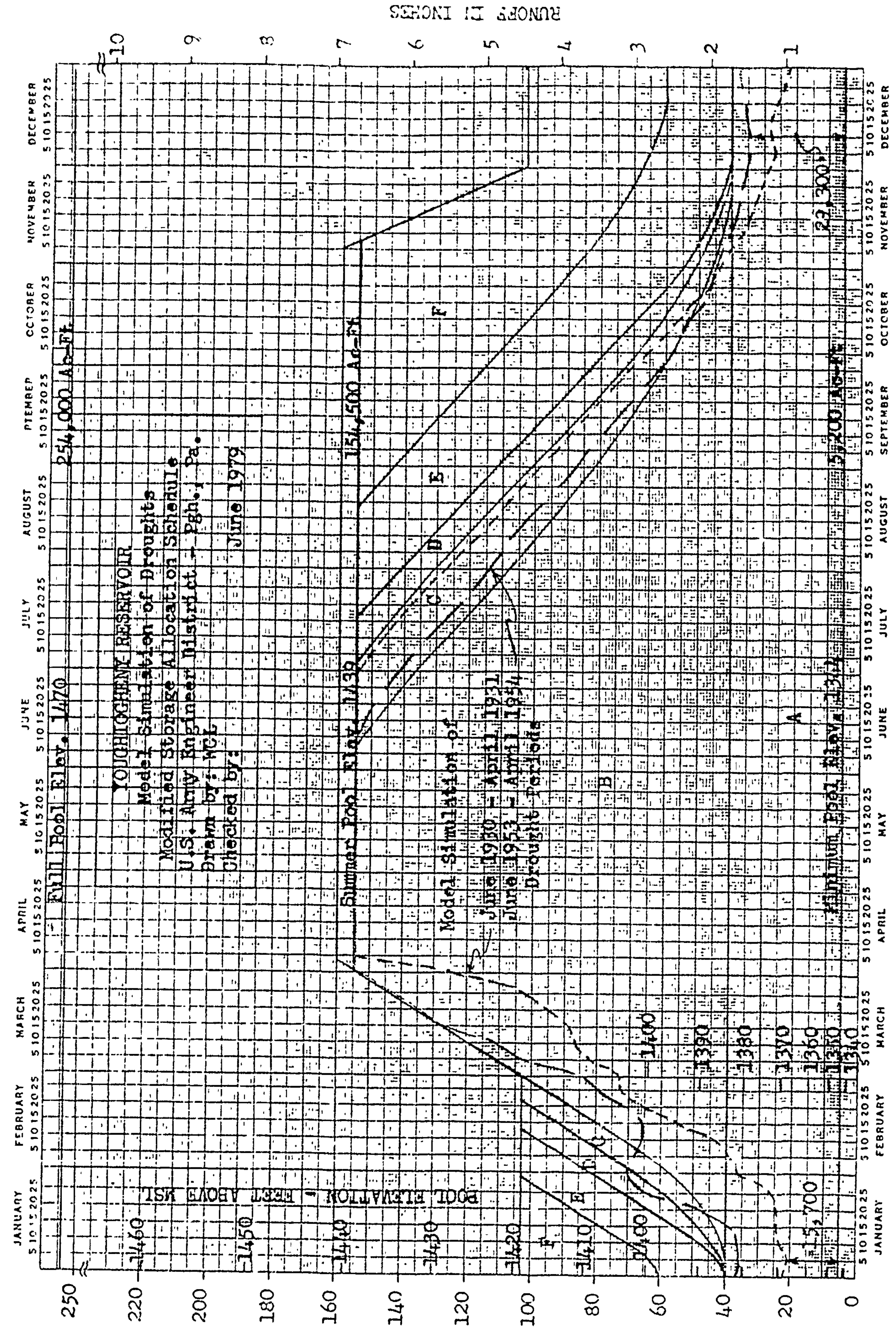
storage for the 1930-31 drought period of 10,000 Ac-Ft that 3,000 to 5,000 Ac-Ft would be available for water supply. Approximately 10 different combinations were tried. Most of the trials provided no better than one-half of the targeted excess amount. Significant excess occurred, however, when Zones A & B were increased by 50 c.f.s. and Zones C & D decreased by 50 c.f.s. This resulted in Zones B & C becoming identical and therefore were combined into one Zone. With guide curves adjusted for these Zones to better accommodate the 1930-31 and 1953-54 drought periods, the result without any structural modification was a computed excess storage of 15,700 and 29,300 Ac-Ft respectively. The model simulations of these two drought periods and the modified release schedules are presented on Plate 8. It should be noted, that the modified storage allocation schedules assumed all reservoir inflow would be allocated to low-flow augmentation. Any contract would, therefore be a purchase of water storage and not of a specific yield.

Using the present storage allocation schedule, the 7-day once in 10-year low-flow as regulated at Connellsville was previously calculated to be 440 c.f.s. If the modified storage allocation were utilized, this flow would be 380 c.f.s.

Using the modified storage allocation schedule a review of actual stream flow records for the years 1929-1977 was made to determine typical reservoir drawdown rates. The net effect of the change is that the reservoir drawdown during a normal or wet year will be 0.5 to 1 foot lower in September and the drawdown in November will be higher by approximately 1 foot. During a dry year, drawdown is delayed by approximately five days in October with a net increase of 2.5 feet in the pool level by the end of November.

<u>Structural Modifications</u>. The possibility of modifying the structure of Youghiogheny River Lake to add additional storage for water supply was also studied. Two specific cases were considered. They were: (a) Increasing the summer pool elevation from 1439 to 1441 and full pool from 1470 to 1471.5; and (b) Increasing the summer pool elevation from 1439 to 1444 and full pool elevation from 1470 to 1474. Case (a) would provide an additional 5,500 Ac-Ft of storage and case (b) would provide an additional 14,500 Ac-Ft. Increasing the full pool level would be necessary to maintain the same flood control effectiveness of the project.

A preliminary investigation of the 1.5 foot raise indicated that there are no particular structural problems. It appears that any reasonable configuration for a 4 foot raise will require a reduction in the crest roadway width from 16 feet to 14 feet and overall crest width from 25 feet to 22 feet. For any raising of


YOUGHIOGHENY RESERVOIR
Model Simulation of Droughts
Modified Storage Allocation Schedule
U.S. Army Engineer District - Pgh., Pa.
Drawn by: WGL
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Full Pool Elev. 1470
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Model Simulation of
June 1930 - April 1931
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Drought Periods
POOL ELEVATION - FEET ABOVE MSL
CAPACITY IN 1000 ACRE-FEET
RUNOFF IN INCHES
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

Plate 8

the crest level, appreciable excavation of the existing embankment material would be necessary to obtain a satisfactory base upon which to compact the additional materials.

The modifications resulting from a two-foot increase in summer conservation level should not be very extensive. However, there would be costs associated with changes to recreational facilities.

Youghiogheny River Lake - Summary and Conclusions. The study of the water supply potential of the Youghiogheny River Lake has shown that there is no surplus water available for such use utilizing the present storage and release schedules. Some of the alternatives investigated did produce surplus water, but not without significant trade-offs. In some cases, downstream flows would need to be drastically reduced which would in turn affect the seven consecutive day once in ten-year low flow. Another alternative that yielded a surplus, delayed the lake's reaching summer pool by several weeks and resulted in severe lake level drawdowns.

In conclusion, the only apparent viable alternative available to provide water supply from Youghiogheny River Lake would be to increase the maximum summer conservation pool level and physically modify the dam structure and the pertinent features to compensate for the related loss of flood control storage.

Reallocation of Reservoir Storage for Water Supply Issues and Impacts

by

Werner C. Loehlein

SUMMARY OF DISCUSSION BY LOREN W. POPE

There was some discussion on the expected lake variations and how local interests had come to accept the varying pool elevations. It was also pointed out that these variations are in steep sided reservoirs and don't represent large changes in the shoreline.

SESSION III
ADVANCED COMPUTER TECHNIQUES

SUMMARY OF SESSION III
ADVANCED COMPUTER TECHNIQUES

prepared by

Gary R. Dyhouse
St. Louis District

Overview

The presentations covered a wide variety of topics, including frequency analysis, dike layout design, flood level sensitivity analysis, the Muskingum-Cunge technique, Corps progress in CADD/GIS and radar-rainfall estimates. Five papers and one panel discussion were featured.

Paper Presentations

Albert G. Holler, Jr., South Atlantic Division, presented a paper entitled "River Basin Modeling for Regulated Flow Frequencies", dealing with the updating and re-definition of flow frequencies of the Savannah River, which is partially regulated by three Corps reservoirs. The study involved a team of Corps and U.S.G.S. personnel to model the entire basin for with and without reservoir conditions. Discharge frequency relationships were developed for the reach from Augusta to Clyo, Georgia, which is under heavy pressure for development. Although the study reached agreement between Federal agencies on regulated and un-regulated discharge frequency relationships, the results are still being questioned by local interests. The paper noted the need for additional work for Bulletin 17B on the evaluation of regulated flow frequencies, and that significant differences in the discharge frequency relationship between Federal agencies can still exist, even when following 17B. Use of expected probability, all or part of historic flood records, rounding or not rounding skew, use of station vs. regional skew values, etc. can result in notable differences in discharge frequency estimates.

Paul K. Rodman, Fort Worth District, described the analysis of "Valley Storage Impacts in the Upper Trinity River Basin". Recent development pressure in the Dallas-Ft. Worth area has led to concerns for the design integrity of the Dallas Floodway. The North Central Texas Council of Governments, consisting of nine cities and three counties, worked with the Corps to analyze and quantify various proposed future development plans. These plans were shown as five future alternatives, which were then modeled to evaluate the effects on future peak discharges as part of a Regional Environmental Impact Statement. Loss of valley storage to levees and floodplain encroachments under the maximum development scenario would result in overtopping the levees by less than the design flood and a significant loss of freeboard under less intense development. With this information, the local governments have supported Corps regulatory requirements within the 404 program to limit adverse effects to flooding from development, and have further adopted policies to prevent this induced flooding outside of Corps regulatory areas. Corps hydrologic and hydraulic analyses were primarily of a sensitivity testing nature, being part of a Reconnaissance Report for the Upper Trinity River Basin. A more-detailed analysis will be included in the Feasibility effort.

Cecil W. Soileau, New Orleans District, presented his paper entitled "An Analysis of Alternative Training Structures in Southwest Pass, Mississippi River". The initial estimate of a river training program at the mouth of the Southwest Pass to minimize dredging requirements for the 45 feet navigational depth was estimated at $47,000,000. Advanced computer modeling, using

WES's TABS-2 package, was incorporated to analyze cost reduction measures. Various dike configurations were modeled, with the resulting best layout obtaining as good a sediment transport rate but costing $15,000,000 less than the original layout. Alternative analyses with TABS-2 were essentially through sensitivity testing without verification or quantified estimates of shoaling, thus minimizing the need for an expensive data collection system.

Gary W. Brunner, Hydrologic Engineering Center, described recent modifications to the HEC-1 program to incorporate "Muskingum-Cunge Channel Routing", the subject of his paper. This method represents a considerable improvement over the original Muskingum routing technique, in that physically-based channel characteristics may be employed to estimate the required values of X and K. Comparisons of the results of Muskingum-Cunge routing to the full equations of unsteady flow using the popular DAMBRK program of the National Weather Service showed almost no difference for most tests. Muskingum-Cunge solutions diverged from those of the more complete unsteady flow equations only when backwater effects were present, or when a rapidly rising hydrograph was introduced into a relatively flat (less than one foot per mile) channel.

Roger Gauthier, Detroit District, discussed his paper entitled "CAD/GIS Intergraph Capabilities". Although use of Computer Automated De [illegible]/Geographic Information Systems in hydrologic work has lagged behind other engineering disciplines, several recent Corps applications were described, including obtaining HEC-2 and TABS-2 data directly from terrain modeling, utilizing GIS to obtain hydrologic parameters for watershed modeling, and determining flooded areas from CAD/GIS data. Use of CAD/GIS data result in consistent modeling, reduced data acquisition costs, a consolidated data base, and improved access/display capability. The main concerns include the continual maintenance of the data base and the quality assurance of data input to the base.

Panel Presentations

Thomas L. Engdahl, Waterways Experiment Station, updated the conference participants on WES's work in "Weather Radar". Originally developed for military hydrology, but now for Corps flood analyses, radar applications are moving closer to becoming an extremely valuable tool for data collection and real-time forecasting. In combination with rain gages, radar rainfall estimates will give a vastly improved picture of rainfall variations in both space and time. A set of software procedures have been prepared to calibrate radar rainfall with on-the-ground data and to determine basin average precipitation. Tests of these procedures in the Rock Island District have shown improvement in the estimation of storm rainfall, compared to estimates based on rain gages alone. Radar rainfall estimates have reproduced storm timing quite well, but tend to somewhat over-estimate actual rainfall. The application of radar-generated rainfall values to hydrologic models for evaluation of its impact on hydrologic forecasting is currently underway.

Carroll E. Scoggins, Tulsa District, presented a paper entitled "Radar Applications", discussing Tulsa District's efforts in utilizing radar to estimate watershed rainfall and the progress of the National Weather Service's NEXRAD system. While some success has been achieved over the years in utilizing the existing radar system to estimate basin rainfall, the incorporation of NEXRAD (next generation doppler radar) promises much more. The NEXRAD system and the procedures for extracting radar data were presented. Although a considerable improvement over the existing system is expected, an evaluation of the accuracy of the NEXRAD radar data will still be required, through comparison and calibration to on-the-ground rain gages. HEC is assisting the District in further developing procedures for direct incorporation into forecasting models, like HEC-1F, for real-time flood forecasting.

RIVER BASIN MODELING FOR REGULATED FLOW FREQUENCIES

by

Albert G. Holler, Jr.[1]

Introduction

This paper describes the development of a regulated flow frequency curve for the Savannah River at Augusta, Georgia. The study is notable because: (a) it involved a cooperative effort between the Corps of Engineers and the U. S. Geological Survey; (b) the period of record is predominantly unregulated, peak flows occurring before completion of three Corps' multiple purpose reservoirs; (c) it used historic flow data which required selecting an accurate threshold; and (d) procedures beyond the scope of Bulletin 17B were needed.

Study purpose

Development along the Savannah River in the vicinity of Augusta, Georgia, is intense and state and local planners need additional site specific flood information to manage the development. The Savannah District, Corps of Engineers, receives numerous requests for flood discharge and elevation information along the river from Augusta to Clyo, Georgia, a distance of 126 miles. Better flow data was needed to provide information to aid in flood plain planning and in reservoir regulation studies. Accordingly, the Savannah District contracted with the South Carolina District of the U. S. Geological Survey to develop a stream flow model of the Savannah River from Augusta to Clyo. The study included flood-frequency analysis which, because flows at Augusta are influenced by the regulation of upstream Corps lakes, required extensive input from the Savannah District to produce accurate flow data. The Hydrologic Engineering Center (HEC) was requested to review the study and to provide a simulation of streamflows to determine the appropriate starting lake elevations for the floods that occurred before the construction of the reservoirs. The completed report has been furnished to the Federal Emergency Management Agency (FEMA) who will determine if revisions to the Savannah River Flood Insurance Study are needed.

Key issues.

The key issues in the study included selection of flow data and the computational method used to determine the regulated flow frequency curve. Data issues included the magnitude of historic floods, the choice of a threshold for historic floods, the treatment of the 1929 floods, and the period of record. Computational methods utilized for the regulated flow frequency curve included the plotting position method and the total probability method.

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[1] Chief, Hydraulic and Coastal Engineering Branch, South Atlantic Division, Atlantic, Georgia

Summary of Primary Findings.

Results of the study were recently published in U. S. Geological Survey Water Resources Investigations Report 90-4024 "Flood Frequency of the Savannah River At Augusta, Georgia" by Curtis L. Sanders, Jr., Harold E. Kubik, Joseph T. Hoke, Jr., and William H. Kirby. The report concludes that the 1 percent exceedance flow at Augusta is 180,000 cubic feet per second (cfs) under the current water control plan for the upstream three Corps multiple purpose projects. Without the projects, the 1 percent chance exceedance flow at Augusta would be 316,000 cfs.

Physical Setting and Available Data

Description of Project Characteristics.

The Savannah River forms the state boundary between Georgia and South Carolina (Figure 1). The total area of the basin is 10,579 square miles. The Savannah River is formed by the confluence of the Seneca and Tugaloo Rivers which begin on the slopes of the Blue Ridge Mountains in North Carolina. The river meanders in a southeasterly direction through the Piedmont Plateau and Coastal Plain. It reaches the Atlantic Ocean near Savannah, Georgia. Three Corps multiple purpose reservoirs (Thurmond, Hartwell, and Richard B. Russell) are located on the Savannah River upstream from Augusta. Thurmond Dam was completed in 1954. It has a total storage of 2,900,000 acre-feet of which 390,000 acre-feet is dedicated to flood control. Hartwell Dam, completed in 1962, has a total storage capacity of 2,843,000 acre-feet of which 293,000 feet of storage is dedicated to flood control. Richard B. Russell Dam has recently been completed and has a total storage capacity of 1,166,000 acre-feet of which 140,000 acre-feet is dedicated to flood control. Most of the water stored in the lakes is for hydropower production and recreation. Plan formulation project benefits are shown on Table 1. Floods of record occurred in the basin prior to completion of the three reservoirs. Recent decades have been marked by drought and water conservation has been a prime water control objective (see Pat Davis paper "Drought Contingency Planning", this Seminar).

TABLE 1
Estimated Annual Benefits in Plan Formulation
(thousands of dollars)

		Annual Benefits			
Project	Year	Hydropower	Navigation	Flood Control	Recreation
Thurmond	1945	$3,085	$201	$ 32	$ 0
Hartwell	1957	5,310	95	178	0
Russell	1969	7,974	0	55	3,805

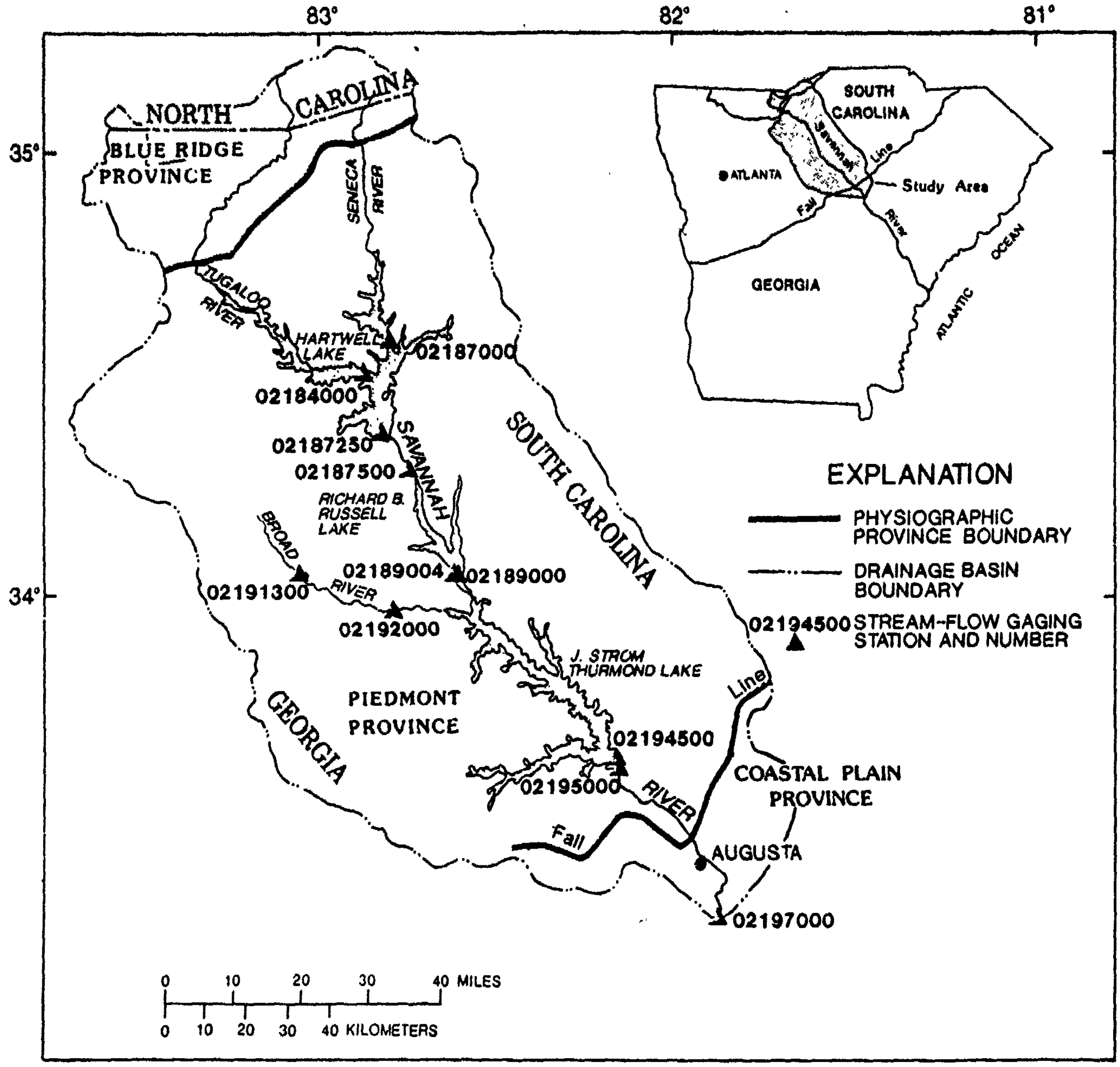

Figure 1. Location of the study area and location of gaging stations in the study area.

(From "Flood Frequency of the Savannah River at Augusta, Georgia", U. S. Geological Survey Water Resources Investigations Report 90-4024, 1990.)

Description of Available Pertinent Data.

Daily stage readings were published for the Savannah River at Augusta from 1875 to the present by the National Weather Service. Since July, 1952, flows at Augusta have been affected by upstream Corps' Reservoirs. The Corps first published discharges for the major historic floods in U. S. House of Representative Executive Documents Numbers 1 and 213. In 1891, the National Weather Service published discharges for major historic floods. The USGS listed the peak stages and discharges for the floods of 1796, 1840, 1852, 1864, 1865, and 1888. In 1951, a USGS publication indicated that discharges for all floods above 225,000 CFS were known. Newspapers and other publications indicate that other floods occurred on the Savannah River at Augusta in 1722, 1741, 1793, 1820, 1824, 1830, 1833, 1851, 1854, 1870, and 1875. The Augusta newspaper began publication in October, 1785, and reported major flooding in the city. In addition, recorded discharges are available for various sub-basins for different periods of time. Peak discharges for historic floods and peak annual discharges from water years 1876-1985 consist of unregulated peak discharges prior to 1952 and regulated peak discharges since 1952.

Study Approach

Procedures adopted. The study utilized the period of record from 1786 through 1985 and included the combined regulation effects of Thurmond, Hartwell, and Russell Dams using the current water control plan. The period of record begins in Water Year 1786 which is the year that the Augusta newspaper began publication. Peak discharges for 1952-85 regulated conditions were converted to natural conditions using a streamflow routing model and inflow hydrographs estimated from daily streamflow and reservoir storage data. The unregulated frequency curve was developed in accordance with procedures in "Guidelines For Determining Flood Flow Frequency - Bulletin 17B" by the Interagency Advisory Committee on Water Data, March 1982. The effect of the reservoirs on floods was determined by selecting nine floods of record for which sub-basin hydrographs were available which could be utilized with the HEC-5 reservoir simulation model. In addition, the hydrograph ordinates were multiplied by 1.25, 1.50, and 2.00 to attempt to define the effect of reservoirs on flows approaching the magnitude of the probable maximum flood. Initial lake elevations were determined using a reservoir routing model and daily flows computed from daily discharge data at gaging stations. The effect of the reservoirs was a function of initial lake elevations and storm size and positioning. This resulted in a wide scatter of points as shown on Figure 2. Regulated flow frequency plotting positions were computed for ten pre-1952 floods that produced the largest regulated flow values. These regulated discharges were either simulated or taken from the regulated-unregulated flow relationship. The regulated frequency curve was defined by averaging the graphical curve fit and the total probability curve.

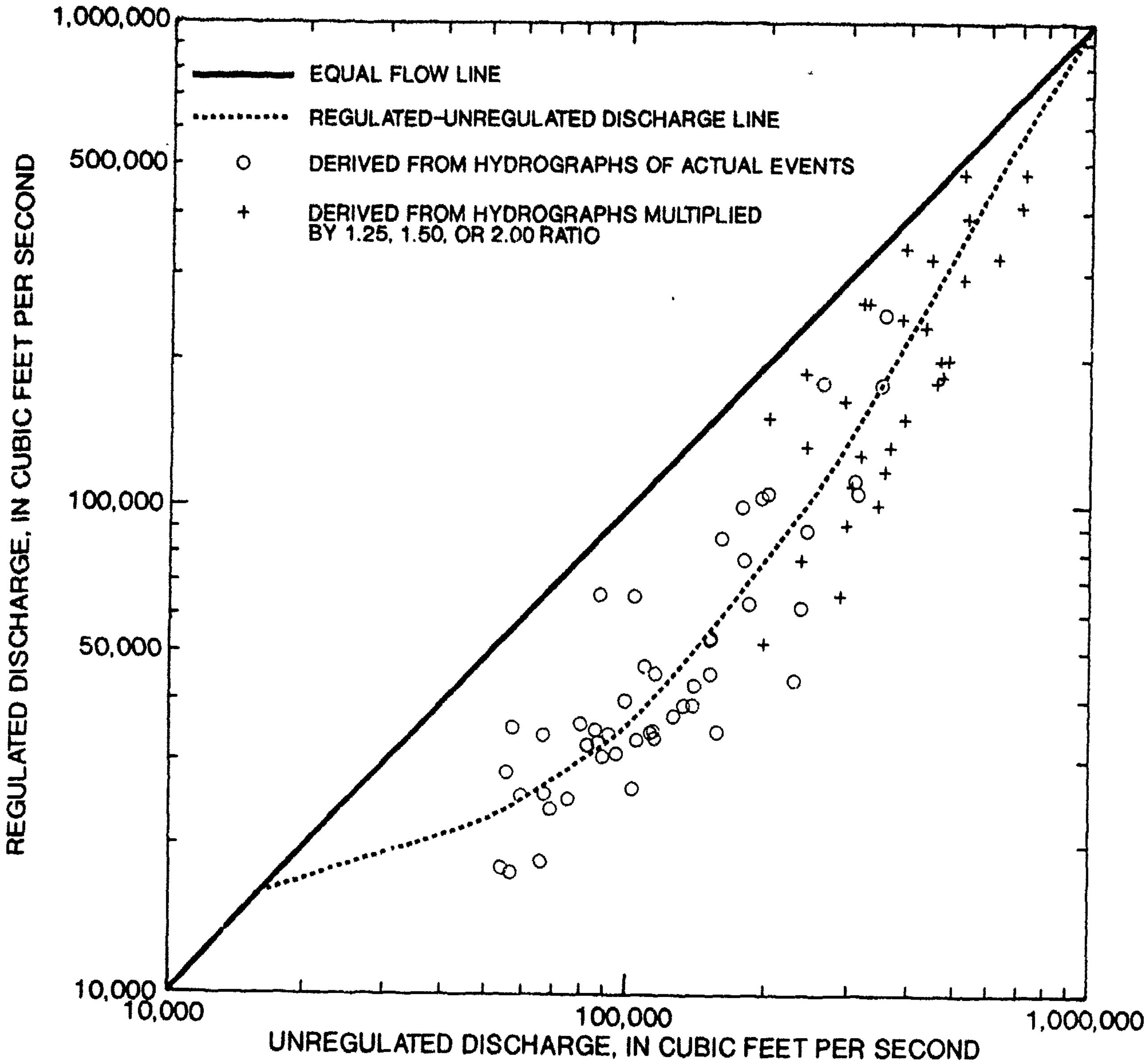

Figure 2. Relation between regulated and unregulated peak discharges for the Savannah River at Augusta, Georgia.

(From "Flood Frequency of the Savannah River at Augusta, Georgia", U. S. Geological Survey Water Resources Investigations Report 90-4024, 1990.)

Key Assumptions and Issues Regarding Project Performance.

The current water control plan includes flood control based upon utilizing five feet of dedicated flood control space within each reservoir while limiting downstream flows to channel capacities (20,000 - 30,000 cfs) and an induced surcharge plan for lake elevations above the top of the flood control pool. Other flood control plans have been suggested which would release larger quantities of water while flood control space is still available within the lakes in the interest of providing more flood control space should larger floods occur. Studies to date indicate no statistically significant differences for the flood control alternative plans. As a matter of fact, if the larger flood does not occur, the proposed flood control plans may cause flooding. And, from an overall water control viewpoint, the suggested plans have the potential to waste valuable water. Weather forecasting has not progressed to the point where it can be effectively used for real time water control purposes on the Savannah River. It takes approximately 24 hours to lower Thurmond Lake one foot for additional flood control storage space by releasing a bankfull flow of 30,000 cfs. During Hurricane Hugo, which produced significant rainfall in some areas, the hurricane switched directions from appearing to head toward Savannah to striking Charleston, South Carolina, in a matter of twelve hours.

Computational methods used.

The natural flow frequency curve was developed by Bulletin 17B procedures. This involved selection of the systematic record, selection of the historic flood events, selection of skew, and selection of period of record. Annual peak flows as published for the period 1876 to 1985 were used for the systematic record. Four historic floods were used in the analysis: 1796 (360,000 cfs); 1840 (270,000 cfs); 1852 (250,000 cfs); and 1865 (240,000 cfs). Normally, a systematic record as long as 110 years would justify the use of the station skew alone. However, for this study a regional skew (-0.1) was weighted with the computed station skew. Some qualitative information ("prodigious flood") is available on floods at Augusta as early as 1722. The Augusta newspaper began publication in October 1785. It appears that the paper reported any major flooding of the city. For this reason, the computational analysis begins in water year 1786.

The peak annual flows for Water Years 1929 and 1930 occurred only five days apart. The floods were caused by two different storm systems and the first flood receded close to channel capacity before the second flood occurred. While it may be argued that the second flood was too close to the first to repeat damages and is not an independent event from a damage standpoint, the two floods were treated as independent hydrologic events for the natural flow frequency analysis. Earlier flow frequency studies by the Savannah District had used a partial duration analysis using flows above a base of 165,000 cfs. However, for the use of historic floods in a

Bulletin 17B analysis, it is critical that all floods above a particular threshold value be known. This is because peak flows in the systematic record that exceed the smallest historic peak are treated as high outliers. Peak flows in the systematic record that are larger than the smallest historic peak are automatically weighted along with the historic peak. It is probable that there were additional unreported floods in the historic period in the 165,000 to 225,000 cfs range. A 1951 USGS report listed all floods in the historic period above a threshold of 225,000 cfs. Therefore, this threshold was used for the study.

Table 2 indicates some possible computed 1 percent chance exceedance natural flow values for the Savannah River at Augusta. The various values are computed by:

- using a threshold of 165,000 rather than 225,000 cfs;
- using a rounded rather than an unrounded skew;
- using expected probability;
- using station skew instead of weighted skew;
- assuming different values for the historic floods;
- considering the Sep-Oct 1929 floods as a single event;
- starting the historic period in 1723 rather than in 1785.

Depending on the assumptions used, unregulated flow values for the 1 percent exceedance event can range from 273,000 cfs to 370,000 cfs. Compiling the most technically supportable set of assumptions produced a value of 316,000 cfs for the 1 percent chance exceedance event.

To compute the regulated frequency curve, the effect of the reservoirs on flows prior to 1952 must be accounted for. There is very limited flood control storage in the projects and it appears to have been provided during project design mainly to replace lost valley storage and to control the more frequent floods. The effect of the reservoirs on floods is a function of: flood volume, flood peak, flood location, and initial reservoir elevation. Regulated discharges were simulated for floods during water years 1908, 1912, 1928, March and September, 1929, 1930, 1936, 1940, and 1949. Table 3 is a summary of the natural peak flows and volumes at Augusta for these floods.

TABLE 2
Computed 1% Chance Exceedance Natural Flows
Savannah River at Augusta

Report*	316,000 cfs
Report, except threshold = 165,000 cfs	312,000
Report, except threshold = 165,000 cfs 1796 = 287,000 cfs	307,000
Report except 1929 = 193,000 cfs	305,000
Report except threshold = 165,000 cfs and 1929 = 193,000 cfs	301,000
Report except threshold = 165,000 cfs 1929 = 193,000 cfs; 1796 = 287,000	295,000
Report except threshold =165,000 cfs; 1929 = 193,000 cfs; 1796 = 287,000 cfs; historic period begins in 1723	273,000
Report except use unrounded station skew	322,000
Report except use rounded station skew	324,000
Report except use rounded station skew and expected probability	334,000
Report except use unrounded station skew; no historic data; expected probability; 1929 = 379,000; 1930 = 387,000 cfs	370,000

*annual natural peak flows 1875-1985; 1929 = 343,000 cfs, 1930 = 350,000 cfs; four historic floods as in USGS records; threshold = 225,000 cfs; unrounded, weighted skew; regional skew = -.1; historic record begins in 1786; computed frequencies without expected probability adjustment; low outlier criterion = 35,000 cfs.

TABLE 3
Pre 1952 Floods Used to Determine the Effect of the Reservoirs on Flows at Augusta

Water Year	Natural Peak cfs	Natural Volume cfs-wk	Peak Vol
1908	307,000	105,900	2.90
1912	234,000	78,800	2.97
1928	226,000	87,500	2.58
1929 (March)	190,400	114,000	1.67
1929 (September)	343,000	127,800	2.68
1930	350,000	215,800	1.62
1936	258,000	154,686	1.67
1940	239,000	101,786	2.34
1949	154,000	81,386	1.89

The regulated-unregulated relationship is shown on Figure 2 and includes data points from the nine storms studied. In addition, actual annual maximum flows measured at Augusta since 1952 are plotted along with a computed value of the corresponding events without the effects of the reservoirs. Also included on the plot are data points resulting from multiplying the nine storm hydrographs by factors of 1.25, 1.50, and 2.00 to define the regulation effects on extreme floods approaching the magnitude of the probable maximum flood.

This relationship between regulated and unregulated peak flows and the unregulated frequency curve were used to determine a regulated frequency curve for the Savannah River at Augusta. Two methods of analysis were used: the plotting position method and the total probability method. The total probability computation is similar to the Corps' coincidental frequency method. Plotting positions were computed by the Weibull formula using a historic period of record of 1786 to 1940 and a systematic period of record of 1952-1985. Results of the plotting position method and the total probability method are shown on Figure 3. Comparison of the two methods indicate good agreement. Therefore the average of the two methods was used to draw a single curve.

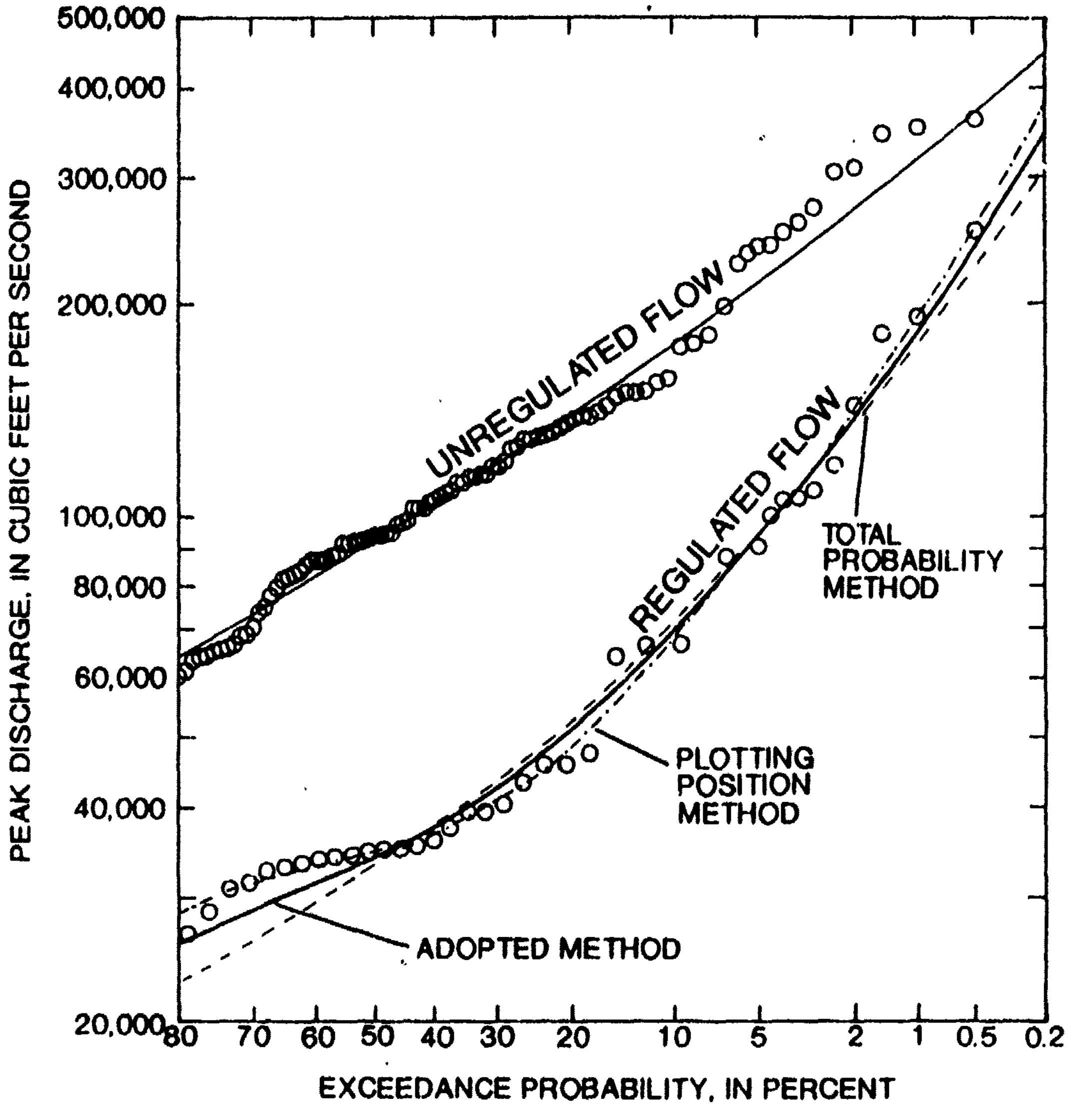

Figure 3. Unregulated and regulated peak discharge frequency curves for the Savannah River at Augusta, Georgia.

(From "Flood Frequency of the Savannah River at Augusta, Georgia", U. S. Geological Survey Water Resources Investigations Report 90-4024, 1990.)

Conclusions

Discussion of conclusions reached regarding project performance.

The results indicate the flood control effects of the reservoirs. Without the reservoirs, bankfull conditions (30,000 cfs) would have a 99 percent annual exceedance frequency. With the projects this flow is reduced to a 50 percent exceedance probability. Other reductions are shown in the table below.

TABLE 4
Computed Flow Frequency Results

Percent Chance of Exceedance	Flows in cfs Natural	Regulated
50	92,000	34,500
20	138,000	51,500
10	174,000	69,000
4	226,000	105,000
2	269,000	140,000
1	316,000	180,000
.5	368,000	240,000
.2	445,000	345,000

Hindsight observations regarding assumptions and procedures use

This study has indicated the need to extend Bulletin 17B procedures to include regulated streams. It has also shown that there remain some minor differences among agencies with regard to computation of a natural flow frequency curve that should be resolved. For example, USGS prefers to use an unrounded skew coefficient while the Corps recommends rounding to the nearest tenth. Also, the flow frequency numbers in this report have not been adjusted for expected probability. While the Corps would recommend making this adjustment, other agencies would not.

This study has also demonstrated the importance of accurately recording and preserving data from episodic hydrologic events. Historic data used in this study was continually challenged by persons other than those with the USGS or the Corps because of the various ways in which it was collected and recorded during an event. While past records and present computations have confirmed the values of the historic floods used in this study, there remains the need to assure that data collected from modern episodic events, such as Hurricane Hugo, become part of a permanent, accurate record. The record should be published for public review and contain the combined data collected by various agencies. The data should be notarized, recorded, and stamped by a professional land surveyor or a professional engineer. The data should survive the ages just as property surveys do because it is just as important.

River Basin Modeling for Regulated Flow Frequencies

by

Albert G. Holler, Jr.

SUMMARY OF DISCUSSION BY GARY R. DYHOUSE

Q: Are the rating curves for the stream gages stable?
A: Yes, little change has been noted for similar discharges over the period of record.

Q: How do the local interests wish to further analyze the regulated flow frequency relationships?
A: Rather than using the largest nine floods in the record and an upward ratioing of these events, the consultant to the local interests suggests basing the frequency determination on running the entire period of record under regulated conditions. Based on only the 110 years of continuous record, this method would result in a lower peak discharge for the 100-year return interval flood. This method would not allow use of the historical data, since volumes and runoff hydrographs are not known. Use of the consultant's method would reduce the Corps/USGS estimate of the 1% chance peak discharge to about 140,000 cfs from 180,000 cfs. Even this value would still inundate essentially all the land in question.

Q: Have you used the Log Pearson III distribution with the regulated conditions?
A: No, this distribution doesn't hold for data that has a significant degree of upstream regulation.

Q: Did you consider the effects of sediment to the reservoirs and the corresponding loss of storage in your results?
A: Yes, but it was negligible. The Savannah River carries a very low sediment load, and what is deposited in the reservoirs will occupy conservation pool storage rather than the flood control pool.

Q: Experiences on the Mississippi River have indicated the strong possibility of overestimating flood discharges when floats are used to measure velocities for stream gaging purposes, as was the case for much of the historical data. Was this considered?
A: No, the recorded discharge data was used directly, along with published estimates of historical flood discharges.

Q: Doesn't development in the reach fall within the wetlands definition of the 404 regulatory program?
A: Yes. No matter what value is adopted for a regulated 1% chance discharge, the wetland designation must be addressed during any development proposal.

Q: Was induced surcharge included during the regulation simulations?
A: Yes.

VALLEY STORAGE IMPACTS IN THE UPPER TRINITY RIVER BASIN

by

Paul K. Rodman [1]

1. Introduction

a. Study Purpose. In 1984 and 1985, boom years in the Dallas-Fort Worth Metroplex real estate market, plans for many new developments in the mainstem Trinity River floodplain came to the attention of the Fort Worth District. Previous studies for the Dallas Floodway system had indicated that future development would reduce the freeboard for the Standard Project Flood (the design flood) for the levees of the Dallas Floodway. Concern about cumulative effects of continued floodplain development and loss of valley storage were primary factors in the decision to conduct a Regional Environmental Impact Study (REIS) for the upper Trinity River basin to develop criteria for making decisions under the Section 404 program. The REIS considered and displayed hydrology, hydraulic, environmental and economic impacts. HEC-1, LRD (Little Rock District hydraulics computer program), and NUDALLAS (Fort Worth District hydrology computer program) were utilized to evaluate various policy alternatives for mainstem floodplain development.

Cumulative impacts of valley storage loss with various development scenarios were shown to be significant. Valley storage loss criteria were adopted for Section 404 permit decisions to limit cumulative impacts. Education of the local cities to the risks of various types of development resulted in their support of our Section 404 permit program and their voluntary support of a regulatory program for the part of the floodplain over which they have jurisdiction and the Corps does not. Problems which were indicated in the REIS caused the North Central Texas Council of Governments (NCTCOG) with 9 member cities and 3 counties to pursue a reconnaissance planning study for the Upper Trinity River.

b. Key Issues. One major issue is the authority of the Corps to require hydrology and hydraulic mitigation in addition to wetlands and fish and wildlife mitigation as part of a Section 404 permit. The tremendously negative consequences of increased upstream and downstream flooding illustrated in the hydrology and hydraulic modelling of various development scenarios generated public support for the Corps to use stringent conveyance (no loss) and valley storage loss (0% for 100-year, 5% for SPF) for development along the mainstem Trinity River. A Geographic Information System (GIS) was used for a rough evaluation of economic impacts.

Another major issue is that of what part of the floodplain should be impacted by the Corp's permitting process. During the REIS and the Reconnaissance Planning Study, the areas requiring a permit were expanded to include abandoned gravel pits and additional areas. The Corps is considering impacts for the SPF floodplain. Federal Emergency Management Agency criteria and the cities traditional procedures regulate development within the 100-year floodplain. As the locals have adopted their own regulatory program (Corridor Development Certificate or CDC), they have expressed significant concern about regulating projects in the SPF floodplain and outside the 100-year floodplain.

c. Summary of Primary Findings. Valley storage along the mainstem of the Trinity River in the Fort Worth-Dallas area is the dominant factor in reducing or increasing peak discharges. Traditional approaches to controlling development in the floodplain result in significantly

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increased downstream flooding, including failure of the Dallas levees for their design flood, the SPF. Education of local public officials and city staff has proved to be essential to gaining support for a "Common Vision" permit program with stringent controls on valley storage and conveyance.

2. Physical Setting and Available Data

a. Description of Project Characteristics. The area hydrologically modeled in this study consisted of the entire drainage area of the Trinity River upstream of the point where Five Mile Creek flows into the Trinity River near the intersection of the Trinity River and Interstate Highway 20 (about 10 miles southeast of downtown Dallas).

The total drainage area at that point is approximately 6,275 square miles. Included in this area is the Fort Worth-Dallas Metroplex. The total drainage areas of the Trinity River at the Elm Fork-West Fork confluence and at the Dallas Gage are 6,061 and 6,106 square miles, respectively. The terrain elevation varies from 1200 feet NGVD at the headwaters of the West Fork of the Trinity River approximately 35 miles south-southwest of Wichita Falls, Texas, to 380 feet NGVD at the confluence of Five Mile Creek and the Trinity River. Figure 1 is a general watershed map for the study area.

Of the five Corps of Engineers flood control lakes in the study area, Lakes Benbrook, Lewisville, and Grapevine were impounded in the early 1950's. The two remaining Corps lakes, Lakes Joe Pool and Ray Roberts, were impounded in January 1987 and June 1987, respectively. Additional Corps of Engineers major flood control projects in the study area include the Dallas Floodway and the Fort Worth Floodway.

The two largest non-Federal lakes in the study area are Lake Bridgeport and Eagle Mountain Lake. Lake Bridgeport is located on the upper West Fork near the city of Bridgeport in Wise County. Eagle Mountain Lake is located in Tarrant County on the West Fork above the much smaller Lake Worth owned by the city of Fort Worth. Eagle Mountain Lake has two sets of gates and an emergency spillway. Since it has no dedicated flood control storage, large and long-duration releases are required during floods. Lake Amon Carter, located in Montague County, is a small lake on Big Sandy Creek north of Lake Bridgeport. Lake Weatherford is a small lake on the upper Clear Fork, located in Parker County. Lake Arlington is a small lake on Village Creek in Tarrant County within the city limits of Arlington. Mountain Creek Lake is a power plant cooling lake on Mountain Creek in Dallas County near the city of Grand Prairie.

The Trinity River watershed is located in a region of temperate mean climatological conditions, experiencing occasional extremes of temperature and rainfall of relatively short duration. The National Oceanic and Atmospheric Administration Station at Fort Worth, Texas shows an average annual rainfall of 32.3 inches during a recent ten year period (1976-1985). The extreme annual rainfall values since 1887 are a maximum of 51.03 inches occurring in 1932, and, a minimum of 17.91 inches occurring in 1921. The mean relative humidity is 65 percent and the average temperature is 65.8 degrees.

Generally the major storms experienced in the study area are produced by heavy rainfall from frontal-type storms which occur in the spring and summer months, but major flooding can also be produced by intense rainfall associated with localized thunderstorms. These thunderstorms may occur at any time during the year but are more prevalent in spring and summer months. Precipitation from hurricane moisture can be very intense and occur over a large area. Hurricane related storms generally occur from July to October.

b. Description of available pertinent data. Some previous Corps of engineers hydrology and hydraulics studies of the Trinity River Basin above the Dallas gage include, the "Definite Project Report on the Dallas Floodway" (1952), the "Comprehensive Survey Report of the Trinity River and Tributaries" (1962), and the "Trinity River Project Memorandum No. 2" (1978).

In addition, hydrology and hydraulics modelling had been performed by the Fort Worth District for many portions of the upper Trinity Basin and the Dallas-Fort Worth Metroplex. Urban studies conducted by the U.S. Geological Survey had been utilized in developing curves indicating impact of urbanization on time to peak (Nelson, 1970) (Rodman, 1977). A real time forecasting model using HEC-1 F was available and had been calibrated for historical events. A NUDALLAS model of the Elm Fork Trinity River below Lake Lewisville was available from a flood insurance study. Design Memoranda or Detail Project Reports were available for the Corps lakes, as well as numerous additional studies conducted since construction.

3. Study Approach

a. Procedures adopted. The area modeled was divided into 108 sub-areas in order to be responsive to the timing of each major tributary's runoff contribution to the total flood hydrograph, and also to obtain detailed flow information (flood hydrographs) at all major points of interest on the West Fork, Elm Fork, and mainstem of the Trinity River. Figure 1 shows the sub-area arrangement. The computer program used to develop the primary hydrologic model for this study was HEC-1. All reservoirs with flood control storage were assumed to be at top of conservation pool level at the start of 2-year to 100-year floods, and at a level corresponding to one-third full flood control pool at the start of the Standard Project Flood. All reservoirs without flood control storage were assumed to be at normal level at the start of all floods.

Separate NUDALLAS hydrology models were developed for the Clear Fork and for the Elm Fork. These models were originally adopted from recent flood insurance studies.

The HEC-1 hydrologic model was first developed for calibration purposes to reflect the 1985 urbanized conditions of the drainage area without Ray Roberts and Joe Pool Lakes. The model was calibrated in reproducing historical flood hydrographs of October 1974, March 1977, October - November 1981, and May 1982. This model was also calibrated by adjusting hydrologic parameters such as "time to peak" and "infiltration loss rates" within reasonable limits in order to match as closely as possible the peak values of five different frequency-floods based on analyses of historical peaks at several streamflow gaging stations. The target values of the peak flows for hypothetical frequency-floods at any particular gage were determined by performing a flow-frequency analysis from the record of flows at that gage. The time period covered by the gage record of flows was selected to start in 1953 (since all major lakes were in place by 1952) and to continue to 1985. This allowed the recorded flows to generally reflect watershed conditions as they were in 1985, before Ray Roberts and Joe Pool Lakes were impounded. The only reservoirs in the study area that were activated after 1952 and before 1985 were Lake Arlington in 1957, Lake Amon Carter in 1956, and Lake Weatherford in 1957. All three are relatively small structures with no flood control storage and generally have a minor influence on the mainstem flow gages downstream. Streamflow gage locations in the study area are shown in Figure 1.

b. Key assumptions and issues regarding project performances. The Dallas Floodway was designed in the early 1950's to handle a Standard Project Flood (SPF) with four feet of freeboard. Hydrology and Hydraulic studies during the 1970's indicated that for ultimate watershed development, the SPF discharge would increase to the point that the levees of the Dallas Floodway would have inadequate freeboard. The accelerated floodplain development of the early 1980's raised concerns about the impacts of projects for which Corps Section 404 (Clean Water Act) or Section 10 (Rivers and Harbors Act of 1899) permits were required. While no one project was big

enough to radically impact the SPF discharges at Dallas, there was significant concern about the cumulative increase in SPF discharge from multiple projects occupying areas which previously stored water and attenuated flood peaks.

c. Computational methods used. The Standard Project Flood (SPF) was developed as outlined in EM-1110-2-1411 (Bulletin 52-8) and distributed in time according to Southwestern Division recommendations. The duration of the Standard Project Storm (SPS) was adopted as 96 hours. The SPS index rainfall was determined to be 14.5 inches. The precipitation amounts used for each sub-area depended on that sub-area's location in the SPS elliptical pattern. The SPS precipitation amounts varied from 5.71 inches for the headwaters sub-area No. 1 to 20.12 inches for sub-area No. 50 (Walker Branch). Due to time and funding constraints only one elliptical storm center, which was the estimated critical center for the Dallas gage, was evaluated.

The hypothetical precipitation for the 2, 5, 10, 25, 50, 100-year and Standard Project Storm was developed using data from National Weather Service Technical Paper 40 (TP40), National Oceanic and Atmospheric Administration Technical Memorandum NWS Hydro-35, and Corps of Engineers Civil Engineer Bulletin No. 52-8 ("Standard Project Flood Determination"). One-hour time increments were used with a 24-hour storm duration for the 2-year through the 100-year storms. Figure 15 of TP40, depth-area-duration curves, was used to adjust the rainfall for watershed size for frequency events. The point rainfall amounts for the 24-hour duration storms for the large area above the various lakes are as follows: 2 year, 3.93 inches; 5 year, 5.30 inches; 10 year, 6.27 inches; 25 year, 7.39 inches; 50 year, 8.38 inches; and, 100 year, 9.38 inches.

The block loss method of estimating infiltration losses was used in this study. Two different loss rates were used: (1) the initial loss which must be satisfied before any runoff occurs and (2) a constant loss in inches-per-hour which continues after the initial loss has been satisfied. The values of both losses vary with the return frequency of the storm. The standard values of loss components for both sand and clay soil corresponding to storm return frequency are as follows:

	Clay Soil		Sandy Soil	
Storm Return Freq.	Initial Loss (IN.)	Constant (IN./HR)	Initial (IN.)	Constant (IN./HR)
2-year	1.5	0.2	.1	0.26
5-year	1.3	0.16	1.8	0.21
10-year	1.12	0.14	1.5	0.18
50-year	0.84	0.10	1.1	0.13
100-year	0.75	0.07	0.9	0.10
SPF	0.50	0.05	0.6	0.08

In the absence of previously determined loss components, the percentage of the watershed with clay soil characteristics and sandy soil characteristics for each sub-area was determined from County Soil Survey Reports published by the U.S. Department of Agriculture. The soil percentages were used to interpolate between the above values to determine the sub-area's loss component values. Where available, loss component values determined from previous studies were used instead of the hypothetical standard values to initiate the calibration process. Comparisons were made between frequency discharges based on analysis of historical data at the major stream gages in the area and the "model computed value of peak flow" at the same gages. Adjustments were made to the loss rates to improve the comparisons of peak flows at the gages. The adjusted values were used in this study. Urbanization and imperviousness were estimated for each subarea.

The imperviousness was not used in the REIS HEC-1 model but was considered in the Reconnaissance Study HEC-1 modelling. Imperviousness was considered in all NUDALLAS modelling. Direct runoff was computed for the impervious area.

Unit hydrographs for the larger subareas above the lakes in the HEC-1 model were generally based on the historical flood hydrograph reproductions for October 1974, March 1977, May 1982 and October-November 1981. Urban curves of tp versus watershed parameters for various percent urban development were used with the smaller, more urban subareas of the HEC-1 model below the lakes. The urban curves were used in the same way for determining tp for the NUDALLAS hydrology model for the Elm Fork. Estimates of the amount of urbanization for each subarea for 1985 Existing Conditions were made by referring to the most recent maps, charts, and aerial photography available. The subarea value of urbanization was assumed to be unchanged from 1985 to 1989 (existing conditions) for the reconnaissance study. Due to the real estate and development recession which has occurred in the Dallas-Fort Worth Area, this assumption is reasonable for most subareas.

In an effort to be as accurate as possible in estimating "percent urbanization" expected to exist as ultimate watershed development (Future Conditions), a request was made by the Corps of Engineers, through the North Central Texas Council of Governments, for information from the major cities in and around the Metroplex as to their projected future development. Thirty-two cities responded in varying degrees to the request for future development estimates, and that information was considered in the estimate of "future conditions" urbanization percentage for each subarea. These estimates for urbanization were used to investigate all future condition alternatives.

Time to peak was developed for each subarea using methodology described in "Synthetic Hydrograph Relationships, Trinity River Tributaries, Fort Worth - Dallas Urban Area" (Nelson 1970). Urbanization curves available for sand (Cross Timbers) and clay (Blackland) soils indicate elapsed time (time to peak) from the midpoint of a unit duration of rainfall to maximum runoff for a given subarea. The geographical characteristics of the subarea such as length of major stream (L), the distance from the subarea outflow point to the location of the subarea center of gravity (Lca), percent urbanization, and the overall slope (S) of the major stream determine the entering arguments for the urbanization curve from which "time-to-peak" for the subarea is extracted. The "time-to-peak" used for each subarea was generated from the Fort Worth-Dallas East-West Cross Timbers Urbanization Curve and the Blackland Urbanization Curve by interpolating between them, based on the percentage of each soil type within the subarea. The percentage of soil type was derived from a Soil Survey report for each county published by the U.S. Department of Agriculture. The East-West Cross Timbers and the Blackland Urbanization Curves are shown in Figures 2 and 3.

The Muskingum routing method was generally used to route through the large subareas of the HEC-1 model above the lakes. Calibration was based on historical flood hydrograph reproductions for October 1974, March 1977, October-November 1981, and May 1982. The modified Puls routing method was used to route through the shorter reaches of the subareas downstream from the lakes. Storage-discharge data were based on HEC-2 and LRD backwater analyses.

4. Study Results.

a. Summary of study results. The Regional Environmental Impact Statement (REIS) involved the North Central Texas Council of Governments (NCTGOG) as convener of representatives of nine cities and three counties having jurisdictional authority for part of the developing floodplain of the Elm Fork and West Fork Trinity River. The U.S. Fish and Wildlife Service, the United States Environmental Protection Agency, and the Federal Emergency

Management Agency (FEMA) were also involved study participants. Five alternative future development scenarios were formulated by the Corps and other study participants in the Draft REIS of 1986 in an attempt to define the range of options. The cities also supplied projections of ultimate watershed development within their jurisdictional areas and outside the mainstem Trinity River floodplain. Hydrology modelling indicates that urbanization of the tributary watersheds without developing or filling the mainstem floodplain results in a small increase in the SPF discharges for the Trinity River at Dallas. Maximum development of the mainstem floodplain using channels and levees to maximize area "reclaimed" results in a radically increased SPF discharge which would overtop the Dallas Floodway levees. Other development options considered included a wider floodway with maximum development, full development of the FEMA floodway fringe, development of the floodway fringe except in areas where a 404 permit is required, and no additional mainstem development with environmental enhancement. Public comment on the Draft REIS resulted in the formulation of additional scenarios. As part of the Final REIS, a composite future was analyzed with limits of mainstem floodplain development defined by local governments staffs. This option represented the most likely development alternative if there were no major shift in regulatory policy. SPF discharge is increased so much that levee overtopping results at Dallas. A theoretical "modified floodway" option was considered with the target of allowing encroachment of the mainstem floodplain so that the 100-year water surface does not increase by more than approximately one foot. Full filling of the floodway fringe in the Draft REIS had resulted in increases in the 100-year and SPF water surface much greater than one foot. The Final REIS was published in October 1987. Table 1 presents REIS SPF discharges and elevations for the Trinity River below the confluence of the Elm Fork and West Fork. Comments were considered from numerous individuals, groups and agencies for consideration in the Record of Decision. District Engineer Colonel Schaulfelberger signed the final Record of Decision on April 29, 1988, formalizing a set of criteria for evaluating environmental and hydrology and hydraulic impacts of projects being evaluated under the Corps Section 404 and Section 10 permit process.

The permit criteria adopted in the Record of Decision evolved during the REIS study period. In August 1986, hydrology and hydraulics (H-H) criteria were written down based on impacts of alternatives evaluated in the Draft EIS and other previous technical studies. Prior to that time, H-H recommendations on 404 permits were made by experienced engineers based on data supplied by developer's engineers and whether "significant negative impacts" resulted locally or cumulatively from the project involved and similar projects. The H-H criteria were further refined in September 1987 to very nearly the same criteria as were adopted in the final Record of Decision. The following H-H criteria for mainstem projects are some of the primary ones included in the Record of Decision: (1) no rise in the 100-year or SPF elevation for the proposed condition; (2) the maximum allowable losses in valley storage (on-site and considering full valley cross-section) are 0% and 5% for the 100-year and SPF, respectively; (3) alterations of the floodplain may not create or increase an erosive water velocity on- or off-site; (4) and, minimum elevation for fills is the 100-year elevation plus one foot, while minimum top of levee is the SPF elevation plus four feet, unless a relief system is designed which prevents catastrophic failure. The H-H personnel of the Fort Worth District Corps of Engineers have spent many hours reviewing data from engineers for developers and discussing and explaining H-H criteria over the last few years in conjunction with evaluating 404 permit requests. Projects which have required Section 404 permits have been designed significantly differently than traditional projects. Significant areas have been set aside for valley storage mitigation, frequently with extensive excavation and wetland creation. Hydraulic impacts have generally been mitigated by relief swales or channel enlargement so that water surface profiles for the 100-year and SPF floods have not been raised significantly.

While the Section 404 permit program offers some limitation of negative impacts of development of the mainstem floodplain, much of the floodplain can be developed without a 404 permit. Uncontrolled development of this part of the floodplain would result in significant

negative impacts. The Corps has continued to meet with the NCTCOG and impacted cities and counties in developing a "Common Vision" whereby the local political entities and the Corps would use common criteria in evaluating floodplain development. The Corps and the cities are working on a Corridor Development Certificate (CDC) process to be followed by developers in seeking permits for any mainstem floodplain development. The Cities have each endorsed a resolution in support of the CDC process. A committee of local government staff representatives is working with Corps personnel on forms and procedures for implementing the CDC process. Much work is yet to be done. A draft manual and ordinance for implementing the CDC process is expected in the next few months.

A Reconnaissance Study for the Upper Trinity River Basin has recently been completed. Several flood control alternatives were recommended for further analysis in a Feasibility Study. The alternative which offers benefits to the entire region is the Boyd Detention Structure. Due to the high cost of land and fish and wildlife mitigation, the Boyd Detention Structure may not be economically justified by conventional Corps' economics. This structure would be located on the

TABLE 1
REGIONAL ENVIRONMENTAL IMPACT STATEMENT (1987)
TRINITY RIVER BELOW CONFLUENCE
OF WEST FORK AND ELM FORK
FOR STANDARD PROJECT FLOOD

Scenario	Discharge (cfs)	Elevation (feet NGVD)
Existing	242,900	433.1
Future Without Mainstem Changes	243,400	433.2
Permits in FEMA Floodway Fringe	284,600	435.7
Modified Floodway	280,400	435.5
Levee Crest Elevation	-	437.2
Composite Future	330,100	438.3
Maximum Development 1, Levees as Close Together as Possible	426,600	443.1
Maximum Development 2, Levees Farther Apart as in Dallas and Fort Worth Floodways	405,500	442.2

West Fork Trinity River downstream of Bridgeport Reservoir with the dam site northwest of the town of Boyd. The drainage areas of Boyd Detention Structure and Eagle Mountain Lake are 1703 and 1970 square miles, respectively. The design considered provides flood control for approximately a 50-year event. In order to quantify the effects of the Boyd Detention Structure or downstream discharges for long-term, system floods, HEC-5 daily models of the 1981 and 1989 floods were built. The HEC-5 analyses indicate significant benefits for the detention structure for long-term, system flood events. The cost of Dam Safety repairs for Lake Worth currently being studied could be reduced or eliminated. Drainage of interior areas behind the levees and freeboard for the levees of the Fort Worth Floodway would be improved due to the detention structure. The reduced discharges and stages due to the Boyd Detention Structure may offer some mitigation for other flood control alternatives of the Reconnaissance Study such as levees which reduce valley storage in protecting local areas.

b. Description of hydrologic engineering results that define project performance. Figure 4 presents the computed Standard Project Flood elevations for the Trinity River below the confluence of the Elm Fork and West Fork from the 1987 REIS. Maximum Development 1 and 2 generate water surface elevations of about 442 feet NGVD (assuming infinitely high levees) The Composite Future scenario results in a computed elevation of 438.3 feet NGVD. The levee crest is at 437.2 feet NGVD. The Modified Floodway scenario generates a water surface of 435.5 which leaves less than two feet of freeboard. Allowing permits in only the (FEMA) Floodway Fringe produces a water surface of 435.7 feet NGVD. Future watershed development (imperviousness was not included in the REIS HEC-1 model) without added changes or development in the mainstem floodplain (Future Without) results in an elevation of 433.2 feet NGVD. For Existing conditions the computed elevation is 433.1 feet NGVD.

In developing discharges for the 1990 Reconnaissance Study, imperviousness was added to the HEC-1 model and mainstem valley storages for Puls routing were updated to the best available data. The rating curve for the Trinity River at Dallas streamgage was adjusted based on U.S. Geological Survey observations and estimates for the 1989 flood. Table 2 presents the discharges and elevations for the SPF below the confluence of the Elm Fork and West Fork. Existing conditions elevation and Future Without are increased to 433.5 feet NGVD and 434.5 feet NGVD, respectively. For Future with CDC with its five percent SPF valley storage loss, the computed elevation increases to 435.1 feet NGVD which is just over two feet below the levee crest of 437.2 feet NGVD. The water surface for the Composite Future is 439.1 feet NGVD which is almost two feet over the top of the levee.

The Composite Future scenario reflects conditions which would have likely occurred without any change in development permitting criteria by the cities. Thus, the policy change of the Common Vision and the CDC process results in a computed water surface for the future which is four feet lower at this critical location on the Dallas Floodway.

5. Conclusions

a. Discussion of conclusions reached regarding project performance. The hydrologic modelling used for this study is neither advanced nor exceptionally complex. The significant accomplishment is the communication of the impact of policy decisions to the decision makers and the subsequent impact on policy. Fort Worth District hydrologic engineers have actively participated and communicated with developers and their engineers, other federal agencies, and the cities at the North Texas Council of Governments for the last six years. We are continuing this effort with participation in implementation of the CDC process. It is anticipated that with the CDC process we will have a technical review function for mainstem floodplain development.

The hydrology models indicated significant cumulative negative impacts from scenarios which allowed maximum development or even most likely future development in the mainstem floodplain. If the CDC process can be rigidly implemented so that only five percent of the SPF valley storage is lost, cumulative impacts are still negative, but at least slightly more than two feet of freeboard remains for the SPF in the Dallas Floodway. To state the results slightly differently, additional channelization within the Dallas Floodway can be economically justified with the Composite Future scenario. With the CDC process, no additional channelization is justified or required.

b. Hindsight observations regarding assumptions and procedures used. The REIS was to some extent a sensitivity analysis. Some rough assumptions had to be made at times. One of those involved totally blocking out all storage in areas considered for future development scenarios in the mainstem floodplain. While this may be reasonable for levied areas, obviously part of the volume above areas filled to the 100-year level will be available for more rare events. We received considerable complaint and criticism from engineers for developers who felt we were overestimating valley storage loss. In fact, we had no definite plans to tell us what type of project would be constructed at any particular location for the development scenarios. It was interesting that part of the complaint was that we had effectively already impacted policy so that our assumptions were no longer in line with what would probably happen.

TABLE 2
UPPER TRINITY RIVER RECONNAISSANCE STUDY (1990)
TRINITY RIVER BELOW CONFLUENCE OF WEST FORK AND ELM FORK
FOR STANDARD PROJECT FLOOD

Scenario	Discharge (cfs)	Elevation (feet NGVD)
Existing	248,400	433.5
Future Without Mainstem Changes	259,600	434.3
Future with CDC, 5% SPF Storage Loss	274,500	435.1
Levee Crest	-	437.2
Composite Future	344,600	439.1

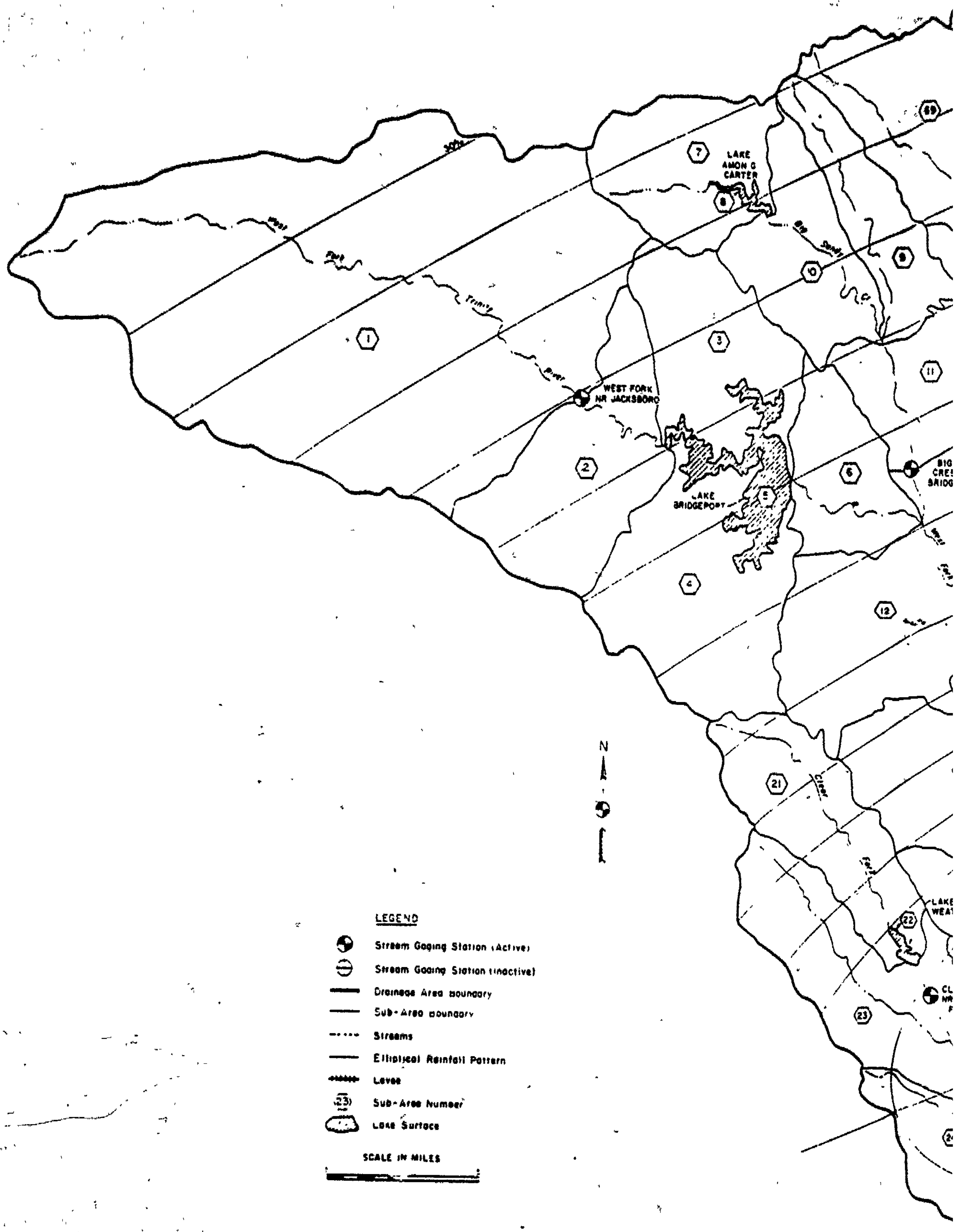
At the beginning of the REIS, we felt that we should evaluate policy criteria which allowed a certain percent reduction in valley storage. The involved cities and other agencies and engineers for developers pushed us to consider alternatives such as maximum development, allowing development in the FEMA floodway fringe, most likely mainstem development considering the cities' plans and desires, and a "modified floodway" option. Looking at all these options was not a direct way to get to a set of criteria for making permit decisions. However, being responsive to the involved cities and other agencies helped educate them to impacts of

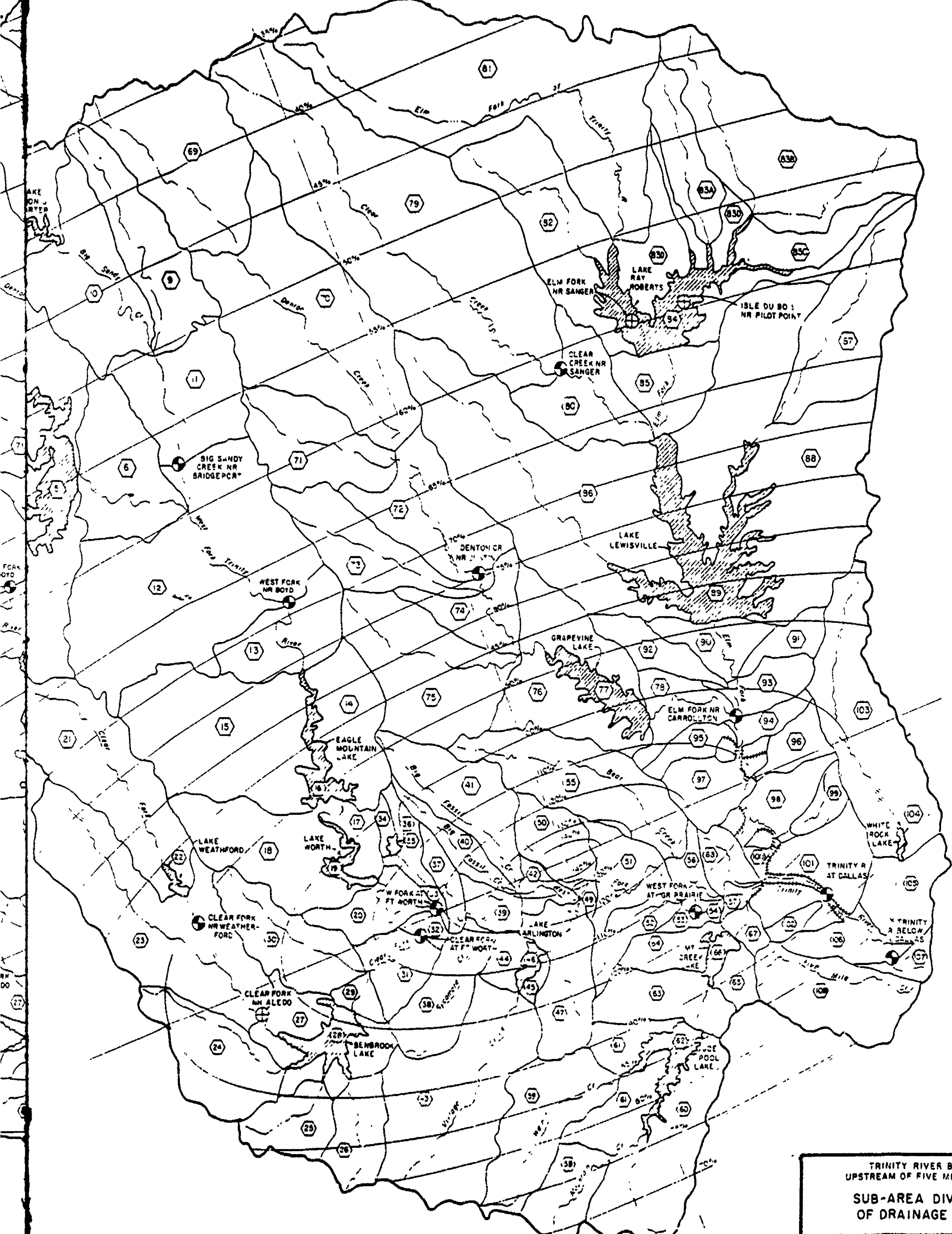
policy decisions and give them a feel for the severity of the situation. Eventually we adopted stringent criteria for Section 404 permit decisions in our Record of Decision. Having gone through this long thought process with us, the cities have voluntarily indicated support for relatively stringent criteria with the CDC program.

References

1. Nelson, T. L., "Synthetic Unit Graph Relationships, Trinity River Tributaries, Fort Worth - Dallas Urban Area," Seminar on Urban Hydrology, Davis, California, 1970.

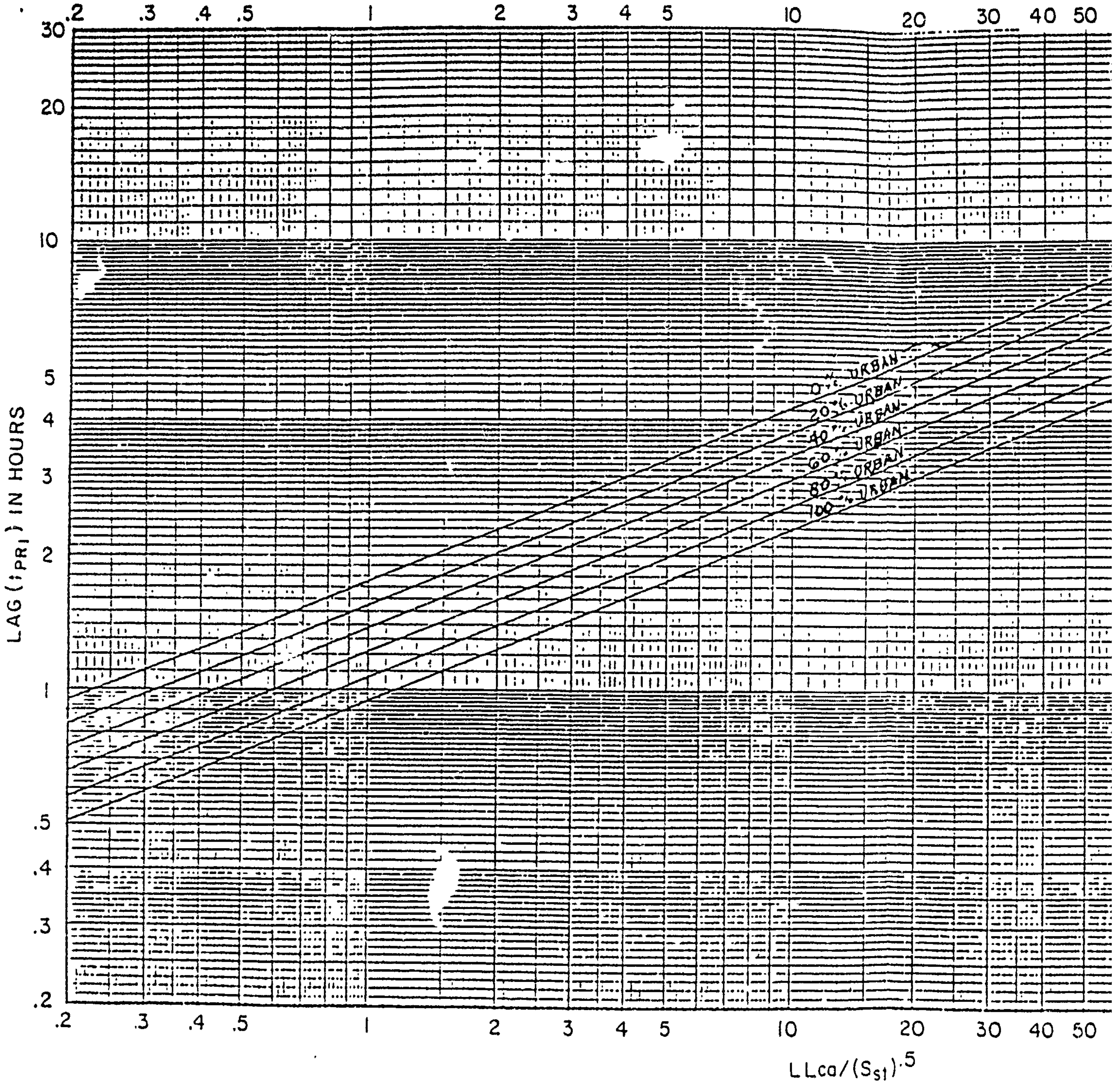
2. Rodman, P. K., "Effects of Urbanization on Various Frequency Peak Discharges," ASCE Fall Meeting Texas, New Mexico and Mexico Section, Albuquerque, New Mexico, 1977.


LAKE AMON G CARTER
WEST FORK NR JACKSBORO
LAKE BRIDGEPORT
West Fork Trinity River
N
LEGEND
Stream Gaging Station (Active)
Stream Gaging Station (Inactive)
Drainage Area Boundary
Sub-Area Boundary
Streams
Elliptical Rainfall Pattern
Levee
Sub-Area Number
Lake Surface
SCALE IN MILES

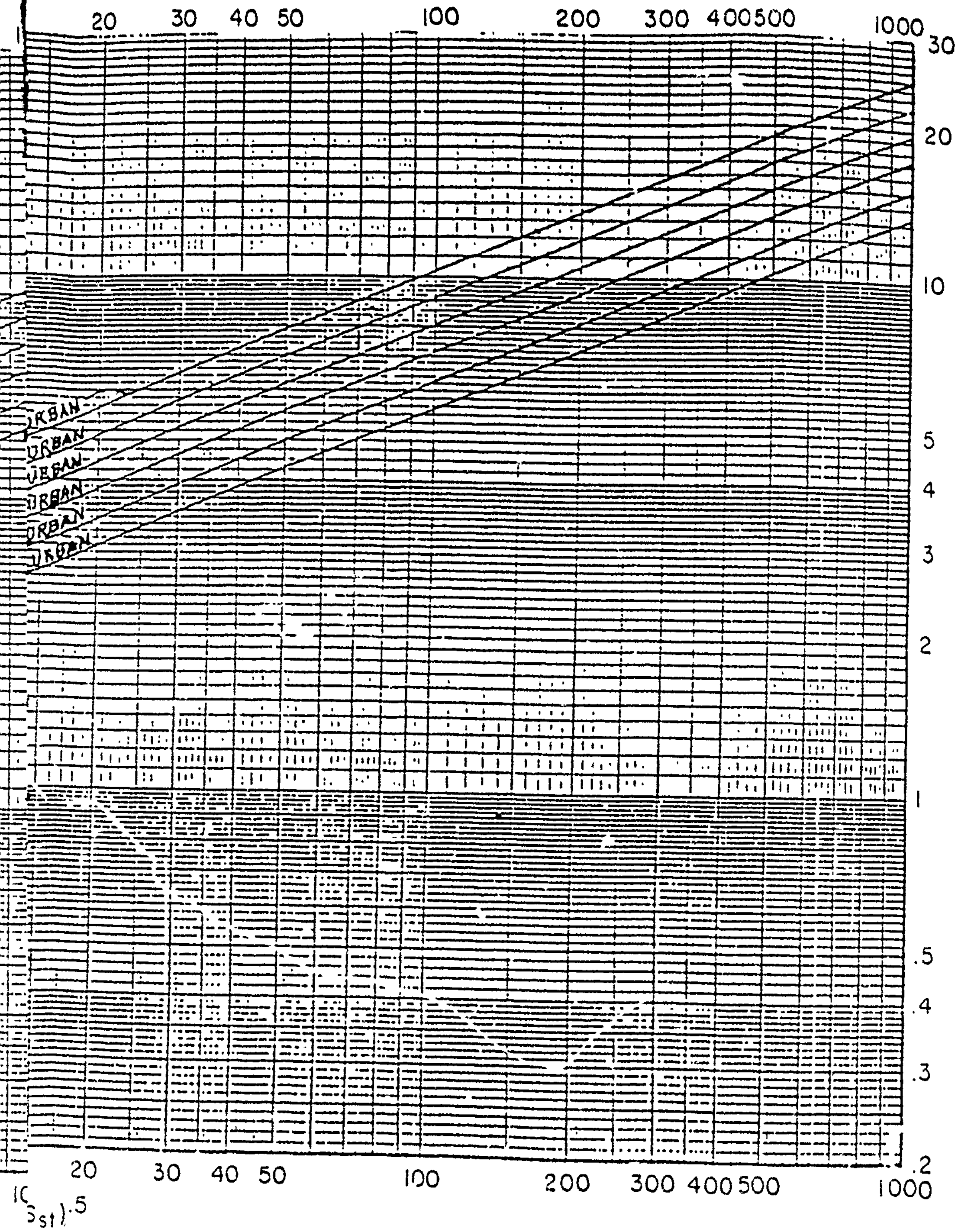


ELM FORK NR SANGER
LAKE RAY ROBERTS
ISLE DU BOIS NR PILOT POINT
CLEAR CREEK NR SANGER
BIG SANDY CREEK NR BRIDGEPORT
LAKE LEWISVILLE
DENTON CR NR JUSTIN
WEST FORK NR BOYD
GRAPEVINE LAKE
ELM FORK NR CARROLLTON
EAGLE MOUNTAIN LAKE
WHITE ROCK LAKE
LAKE WEATHFORD
LAKE WORTH
TRINITY R AT DALLAS
W FORK AT FT WORTH
WEST FORK AT GR PRAIRIE
CLEAR FORK NR WEATHERFORD
CLEAR FORK AT FT WORTH
LAKE ARLINGTON
TRINITY R BELOW DALLAS
CLEAR FORK NR ALEDO
BENBROOK LAKE
JOE POOL LAKE
TRINITY RIVER BASIN
UPSTREAM OF FIVE MILE CREEK
SUB-AREA DIVISION
OF DRAINAGE AREA
U.S. ARMY ENGINEER DISTRICT, FORT WORTH

FIGURE 1



0% URBAN
20% URBAN
40% URBAN
60% URBAN
80% URBAN
100% URBAN
LAG (t_{PR1}) IN HOURS
$LLca/(S_{s1})^{.5}$

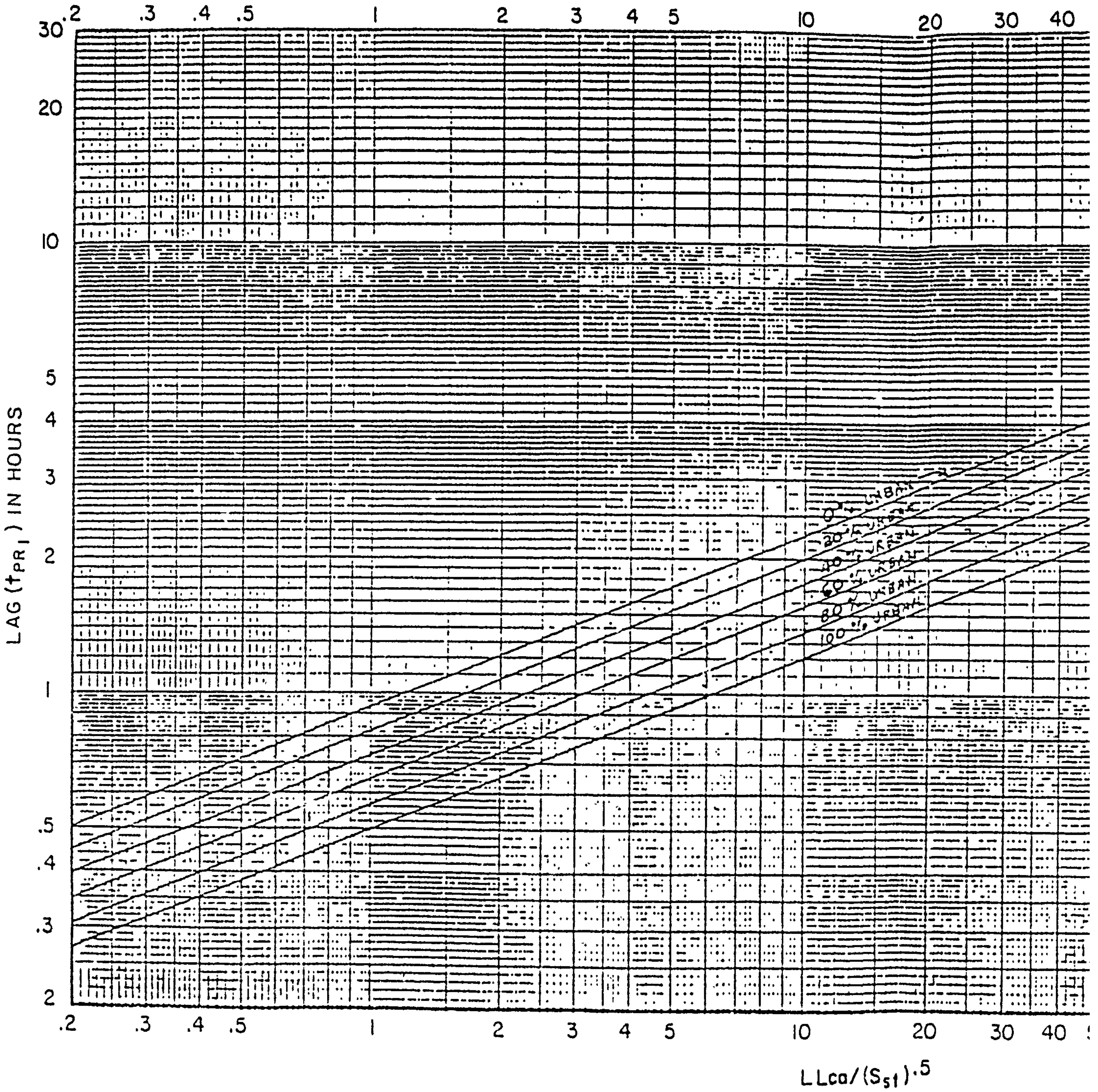


TRINITY RIVER BASIN UPSTREAM OF FIVE MILE CREEK

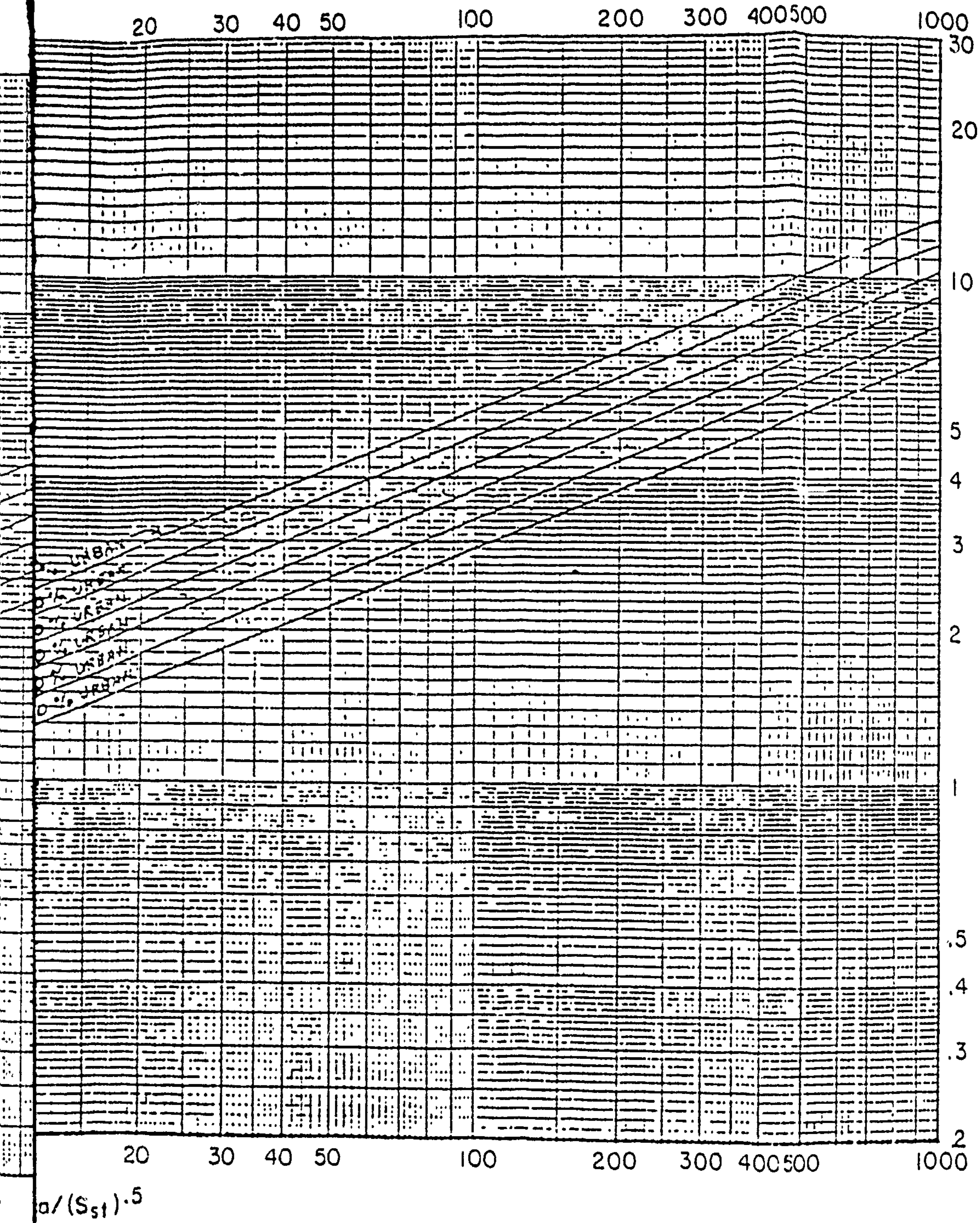
EAST-WEST CROSS TIMBERS URBANIZATION CURVE

U.S. ARMY ENGINEER DISTRICT, FT. WORTH

FIGURE 2


LAG (t_{PR_1}) IN HOURS
$LLca/(S_{st})^{.5}$
0% URBAN
20% URBAN
40% URBAN
60% URBAN
80% URBAN
100% URBAN

U.S. ARMY



TRINITY RIVER BASIN UPSTREAM
OF FIVE MILE CREEK

BLACKLAND PRAIRIE
URBANIZATION CURVE

U.S. ARMY ENGINEER DISTRICT, FT. WORTH

FIGURE 3

REGIONAL ENVIRONMENTAL IMPACT STATEMENT 1987

COMPUTED STANDARD PROJECT FLOOD ELEVATIONS FOR TRINITY RIVER BELOW CONFLUENCE OF ELM FORK AND WEST FORK

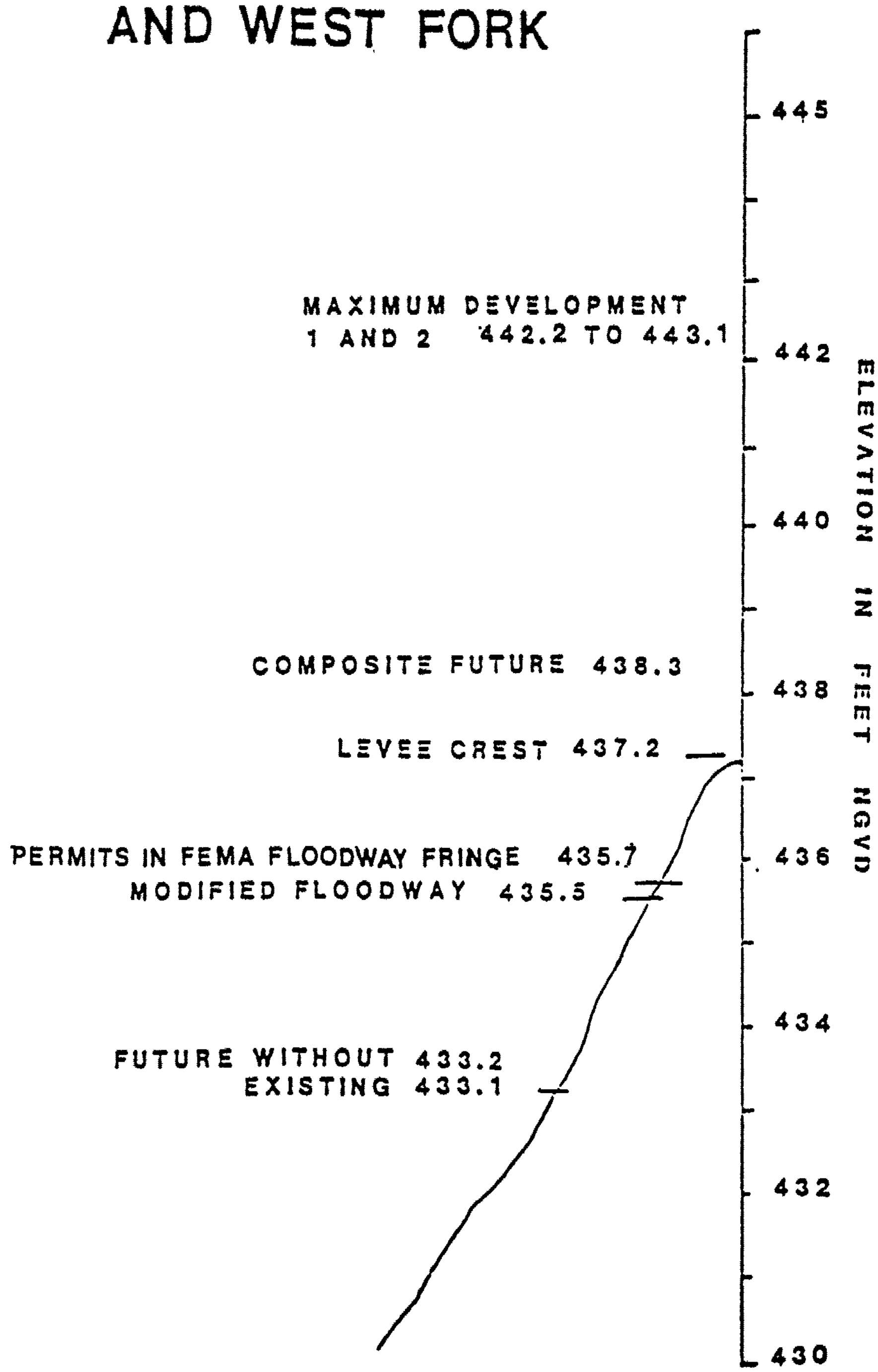


FIGURE 4

Valley Storage Impacts in the Upper Trinity River Basin

by

Paul K. Rodman

SUMMARY OF DISCUSSION BY GARY R. DYHOUSE

Q: If no loss of storage is to be allowed, are the cities/Corps stopping all development in the flood plain or flood fringe?
A: No, development may be allowed with mitigation included, usually in the form of excavated storage to compensate for the loss of storage and conveyance.

Q: Did the 1990 flood threaten the Dallas levees?
A: Not really. Preliminary estimates of this flood are that it was a 40-50 year recurrence interval, compared to the SPF design for the levees. Approximately nine feet of levee freeboard existed at the crest of the flood.

Q: Do local developers have to perform hydrologic analyses for their proposals and are these comparable with Corps hydrologic analyses?
A: Yes, but they are usually less conservative than Corps methods. The developers have generally been co-operative with Corps methods and requirements though.

Q: How was the valley storage loss computed?
A: Modified Puls routing in the HEC-1 or NUDALLAS programs. The LRD water surface profile program was used to develop with and without storage loss values for the watershed modeling programs.

Comment: It appears that valley storage loss must be prevented to maintain the integrity of the Dallas Floodway. This requirement may be necessary to formalize as part of any Upper Trinity River Project. This is also an excellent example of a service the Corps of Engineers can provide to local governments. We should search for other areas and opportunities throughout the country.

AN ANALYSIS OF ALTERNATIVE TRAINING STRUCTURES IN SOUTHWEST PASS, MISSISSIPPI RIVER

by

Cecil W. Soileau[1]

INTRODUCTION

Study Purpose. The Navigation Project, Mississippi River, Baton Rouge to the Gulf of Mexico, La., is the main entrance into the Mississippi River for maritime shipping calling on the ports of New Orleans and Baton Rouge. See Figure 1 (Heltzel, 1989). A deep draft project is maintained from the edge of the Continental Shelf into Southwest Pass, a major distributary of the Mississippi River. The entrance to Southwest Pass from the Gulf of Mexico is guarded by stone jetties which converge from a width of 3600 feet at the shoreline to an opening of about 1500 feet at the ends 3.5 miles offshore. The modeling effort I describe here was intended to find suitable alternatives to an expensive rehabilitation program for the Inner Bulkhead, a feature of the jetty reach of the Pass depicted by a dashed line on Figure 5. The Inner Bulkhead (Corps of Engineers, 1984) was intended to rectify the hydraulic problem created by the jetties having been constructed at a distance too far apart in 1904. Over time the bulkheads settled out of site into the mud line and ceased to provide the required confinement for efficient sediment transport in the reach. The shoaling problem has worsened with time as the project depth has changed from 30 feet to 45 feet in 5-foot increments between 1930 and 1988.

Key Issues. The original construction consisted of two parallel and continuous Wakefield type timber walls each 5 miles long which were constructed in the early part of this century. Over time, due to high subsidence rates, the walls, which were constructed to a height of 6 feet above Mean Low Water, slipped below the water level even at low tide and provided little hydraulic benefit to sediment transport. The poor channel conditions and higher cost of maintenance associated with the poor hydraulic efficiency, caused the New Orleans District during the 1973 flood on the Mississippi River, to seek ways to increase transport of sediments through the reach. Lateral timber-pile dikes had been tried as an initial replacement for the parallel bulkhead in 1937 but only slight improvement was realized and improvement was short-lived because the project was deepened to 40 feet Mean Low Gulf in 1941. With time the bulkhead completely disappeared and it was decided that it needed to be replaced.

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[1]Supervisory Hydraulic Engineer, New Orleans District, U.S. Army Corps of Engineers

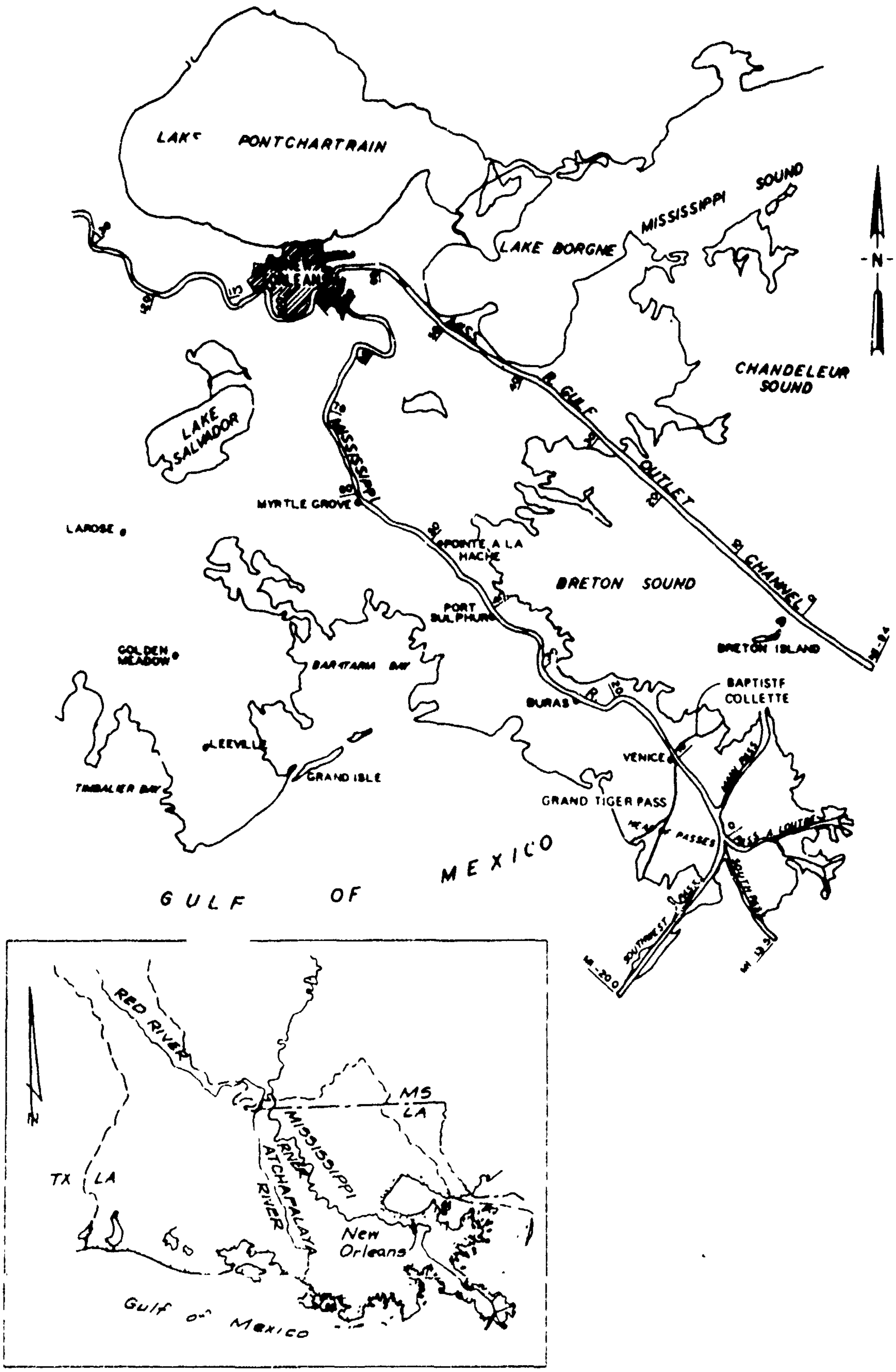


Figure 1. Location map

A 54-inch hollow Raymond Concrete Pile wall was selected to replace the Wakefield type bulkhead in this reach at an estimated cost of $16 million, or about $5 million a mile. A rather significant expense which triggered the question: Can the same benefit be achieved some other way? An extensive design study was initiated to find alternate methods of construction more suited to the soft foundation conditions which are found in southeast Louisiana. Rock dikes with clam shell core on a geotextile proved to be the cheapest alternative to the Wakefield type bulkhead in shallow water, about $1 million a mile, but were too expensive in deeper waters of the lower 1.7 miles of the Pass, about $6 million a mile due to their greater structural height. The Waterways Experiment Station was asked to model the Pass and evaluate the sediment transport efficiency of the proposed bulkhead against other training works.

Summary of Primary Findings. The Waterways Models (Heltzel, 1989) TABS-2 and StudH were used to test seven alternative plans, A through G, against the base condition (Figure 2), to determine the most effective dike configuration and least costly alternative that would provide comparable benefits to sediment transport. The Plan E lateral dike enhancement plan proved to be as effective as Plan G, the Concrete Pile Bulkhead alternative, at about one-third the cost of the recommended plan.

PHYSICAL SETTING AND AVAILABLE DATA

Description of Project Characteristics. The deep draft project of the Mississippi River (Figure 1) is maintained at a depth of 45 feet Mean Low Water with advanced maintenance dredging required to 52 feet Mean Low Water during spring and summer when the river flow may be as high as 1,250,000 cfs. Southwest Pass will carry 500,000 cfs of the flow, or 42 percent of the flow at a velocity of 4 to 5 feet per second. The average annual sediment transport of the Mississippi River is about 1,650,000 tons, of which 5 to 20 percent is fine sands and the remainder is comprised of silts and clays. About 33,000,000 tons must be dredged from Southwest Pass annually and about one-half of that is dredged from the 5-mile-long Jetty Reach. The shoals in the reach form as fluff due to flocculation and consolidate over time into low density-fluid mud. This fluid mud frequently interferes with ship steerage and foregoes any ship passing ship on this segment. The location of the flocculation varies with tide phase and river discharge in the Pass. When the discharge is about 300,000 cfs, the shoals occur in the first mile of the entrance channel. When the discharge is 250,000 cfs, the shoals occur some 20 miles upstream at Head of Passes (Figure 1) where the three major distributaries join to make the main channel.

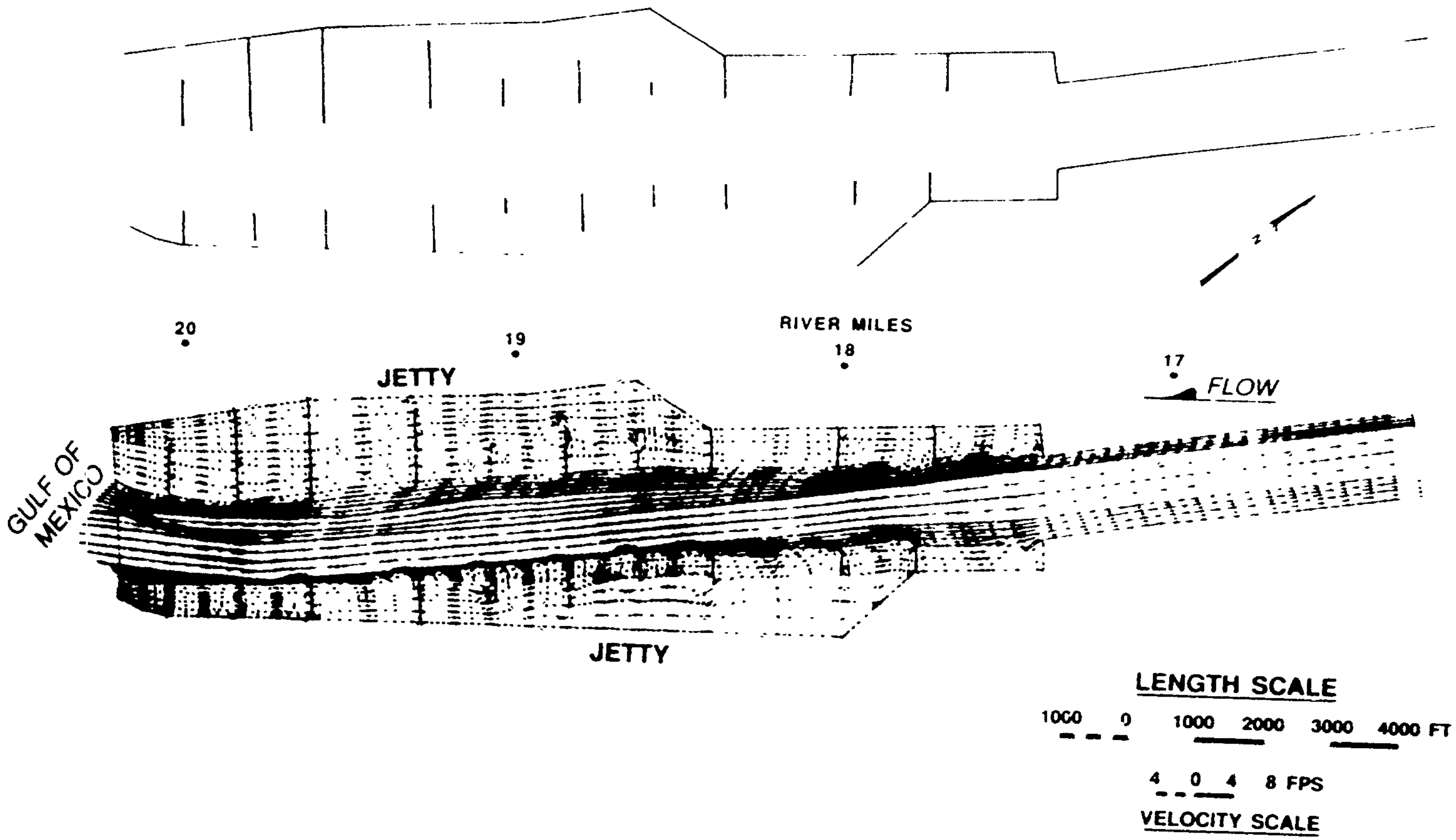


Figure 2. Base dike layout and velocity field

Description of Available Pertinent Data. Hydrographic surveys are done several times each week, particularly during the dredging season, and provide a useful indication of shoaling rates although the data is imperfect due to the constant need of dredging for navigation. River stages may range as high as 4 feet above Mean Sea Level (0.5 feet NGVD) during floods at the jetty reach, 20 feet at the port of New Orleans, and 45 feet at Baton Rouge. The river discharge may reach 1,500,000 cfs at Baton Rouge during floods. Tides in the Gulf of Mexico have little effect on the river at such large discharges. Major floods occur about every 7 to 10 years on the Mississippi River and dredging efforts in Southwest Pass due to these events can exceed 60,000,000 cubic yards in the flood year and 40,000,000 cubic yards in the following year, primarily because the dredging fleet is inadequate to handle the sediment load in 1 year of dredging. The normal range of tide is 1 foot at the jetties, and during low river discharges of 250,000 cfs, tidal variation 120 miles upstream at New Orleans is 0.8 feet and 240 miles upstream at Baton Rouge is 0.2 feet. Wind tides due to extra tropical storms may range between 2 to 6 feet, and hurricane surges have been observed as high as 16 feet Mean Sea Level in the vicinity of Southwest Pass. Shoaling, due to tidal effects, is not observed for river discharges greater than 900,000 cfs. Hurricane waves can reach a height of 15 feet at the jetties and can contribute significantly to dredging requirements due to offshore sediments which are transported into the navigation channel over river banks along with sea water. The high salinities associated with the sea water inflow can cause immediate flocculation and deposition of silts and clays over the entire length of the river below New Orleans. As a consequence, shoaling rates associated with hurricanes can be as high as those associated with riverine floods and the magnitude may approach 60,000,000 cubic yards in one event.

STUDY APPROACH

Procedures Adopted. A study plan was developed by WES and New Orleans District to use numerical models to evaluate the relative merit of the recommended bulkhead plan and six alternative lateral dike geometries against the base condition in the jetty reach of Southwest Pass. Table 1 (Heltzel, 1989) gives a brief description of these alternatives. The main idea was to salvage whatever benefits were being realized from the existing lateral dike fields in the Pass and then enhance these by building additional lateral dikes in the reach to achieve the same effect that is provided by the bulkhead plan. The most recent hydrographic surveys and design drawings (Corps of Engineers, 1984) of the existing dikes and proposed bulkhead wall were used to construct the eight desired geometries into Finite Element meshes. The TABS-2 numerical modeling system was then

TABLE 1

Alternatives Tested

Alternative	Features	Comments
Base	Existing dikes	With existing training works in place
Plan A	Double the existing dikes	Not all connected to jetties
Plan B	5 dikes connected to jetty	All on right bank
Plan C	5 left bank, 5 right bank dikes connected to jetties	Otherwise, same as Plan A
Plan D	3 left bank, 1 right bank dikes connected to jetties	Otherwise, same as Base
Plan E	4 left bank, 2 right bank dikes connected to jetties	Otherwise, same as Base
Plan F	3 right bank dikes connected to jetty	Two base dikes, one additional lateral dike
Plan G	Inner continuous bulkhead	General Design Memorandum Plan

applied to each mesh to make qualitative comparisons of the hydrodynamics between the different alternatives. Then 2-D Vector Plots of velocities were developed for comparisons between tests. Sediment transport capacities were determined by analytical methods using a modified Colby relationship (Heltzel, 1989). This approach provided additional insight into how each alternative would perform.

Key Assumptions. Two key assumptions made for this study were as follows:

1) If a high flow rate, 900,000 cfs, in the Mississippi River were applied to the TABS-2 model, the impact on shoaling rates due to gulf tides would be immeasurable and the solution would become one of steady state.

2) Since all conditions to be tested have the same geometry and depth, and differ only in the configuration and number of lateral dikes, the model did not have to be verified or have to provide quantitative shoaling results to be useful as a screening tool.

Other pertinent assumptions made for this study are given in Table 2.

Computational Methods. The TABS-2 numerical modeling system can be used to study two-dimensional hydrodynamics, sedimentation, and transport problems in rivers and estuaries. It was used here as a stand-alone solution technique to compute water-surface elevation, velocity, current patterns, sediment erosion, transport, deposition, bed change, and hydraulic feedback. Velocities were compared along the centerline of the jetty reach, adjacent to existing and proposed dikes, and midway between the dike spacing. A weighted average velocity and sediment transport rate was calculated for each dike location. Velocities at locations adjacent to the dikes were given a weight of 1/6 and the velocity on the centerline of the channel was given a weight of 2/3. Water-surface elevations calculated by the RMA-2 were also compared and found to vary by no more than 0.05 foot between all geometries studies. Because the model was not verified, sensitivity testing of StudH was carried out to determine those model parameters that were relatively important in the analysis. It was found that a dispersion coefficient less than 0.20 m2/sec caused sediment concentration oscillations and StudH could not be made to produce reasonable results. For this reason, StudH was not used any further, and the Colby's sediment transport relationships were selected to provide a measure of the benefits of each of the eight plans tested (Heltzel, 1989).

TABLE 2

Hydrologic Parameters

Parameter Description	Value
Manning's roughness n	0.020
Effective particle diameter	0.15 mm
Effective settling velocity	0.0075 m/sec
Upstream boundary concentration	150 mg/l
Percent of river discharge in Pass	19 percent
Discharge in jetty reach	162,000 cfs
Dispersion coefficient	0.20 m2 /sec

STUDY RESULTS

Summary. The study results can be broken down into velocity results and sediment transport results. Table 3 gives results for the base and Plans C, E, and G. The weighted average velocities and sediment transport capacity at each dike is referenced to a mile marker measured below Head of Passes where the main river channel splits into the three major distributaries. Figures 2, 3, 4, and 5 show the 2-D Velocity Vector Plots and the dike/bulkhead layout. The dashed lines on the layouts represent the additions to the base condition. For example, Figure 4-Plan E, without the dashed lines, would represent the base layout. Plan A caused the smallest change in velocity because it allowed the largest amount of flow around the root end of the dikes which were not tied to the jetties, and Plan G caused the greatest change in velocities because it cutoff all flow around the root end of dikes. (Results for Plans A, B, D, and F are not presented for brevity). Plan C caused velocities to be nearly identical to those of Plan G, but Plan E velocities were only slightly lower than Plan G. See Figures 2, 3, 4, and 5. All other dike plans were less effective. The cumulative sediment transport capacities are given in Table 4 for each of the plans. Although the incremental sediment transport potential varied along the channel in each plan, the cumulative transport was greater for Plans C, E, and G. Plan E required the smallest number of dikes to be added to the base condition and provided the same increase in cumulative sediment transport capacity as the recommended Plan G.

CONCLUSIONS

Discussion. The deepening of the entrance channel to 45 feet in the Southwest Pass of the Mississippi River in 1988 (New Orleans District Riverside Review 1990) and the capability to maintain that entrance channel to 45 feet for 100 percent of the time through a combination of hydrologic and geotechnical engineering, and wise scheduling of advanced maintenance dredging, has enabled the Port of New Orleans to regain its position as the nation's number one port in 1989. The deeper channel allowed for larger and more efficient bulk carriers to carry greater amounts of cargo. The largest volume commodities moved in the ports of New Orleans and Baton Rouge were coal, lignite, corn, crude petroleum, soybeans, and fuel oil. The Port of Baton Rouge ranked fifth in the nation in 1989. The use of advanced computer techniques associated with the TABS-2 numerical modeling system, which includes data conditioning, mesh generation, and output presentation, among other labor-saving attributes, has greatly reduced the time associated with screening complex solutions to difficult hydrologic problems and

TABLE 3

Weighted Velocities and Sediment Transport

River Mile	Average Weighted Velocity, fps	Incremental Sediment Transport, tons/day	Cumulative Sediment Transport, tons/day
		Base	
17.80	3.2	65,000	65,000
18.02	3.5	96,000	161,161
18.40	3.3	79,000	240,000
18.62	3.3	81,000	321,000
18.83	3.3	73,000	394,000
19.05	3.3	73,000	467,000
19.27	3.3	79,000	546,000
19.59	3.4	86,000	632,000
19.80	3.6	103,000	735,000
20.00	3.6	111,000	846,000
20.14	3.6	108,000	954,000
		Plan C	
17.80	3.3	75,000	75,000
18.02	3.4	88,000	163,000
18.40	3.4	91,000	254,000
18.62	3.5	94,000	348,000
18.83	3.4	85,000	433,000
19.05	3.4	85,000	518,000
19.27	3.4	86,000	604,000
19.59	3.4	89,000	693,000
19.80	3.6	104,000	797,000
20.00	3.7	114,000	911,000
20.14	3.6	112,000	1,023,000
		Plan E	
17.80	3.2	65,000	65,000
18.02	3.4	88,000	153,000
18.40	3.4	88,000	241,000
18.62	3.5	94,000	335,000
18.83	3.3	80,000	415,000
19.05	3.4	85,000	500,000
19.27	3.4	84,000	584,000

River Mile	Average Weighted Velocity, fps	Incremental Sediment Transport, tons/day	Cumulative Sediment Transport, tons/day
19.59	3.4	88,000	672,000
19.80	3.6	104,000	776,000
20.00	3.6	111,000	887,000
20.14	3.6	109,000	996,000

Plan G

17.80	3.2	65,000	65,000
18.02	3.4	87,000	152,000
18.40	3.4	87,000	239,000
18.62	3.5	92,000	331,000
18.83	3.4	82,000	413,000
19.05	3.4	87,000	500,000
19.27	3.4	84,000	584,000
19.59	3.4	90,000	674,000
19.80	3.6	109,000	783,000
20.00	3.5	101,000	884,000
20.14	3.6	112,000	996,000

TABLE 4

Sediment Transport Results

Plan	Change in Cumulative Sediment Transport Capacity, percent
Base	0.00
A	+1.00
B	+3.00
C	+7.00
D	+2.00
E	+4.00
F	+2.00
G	+4.00

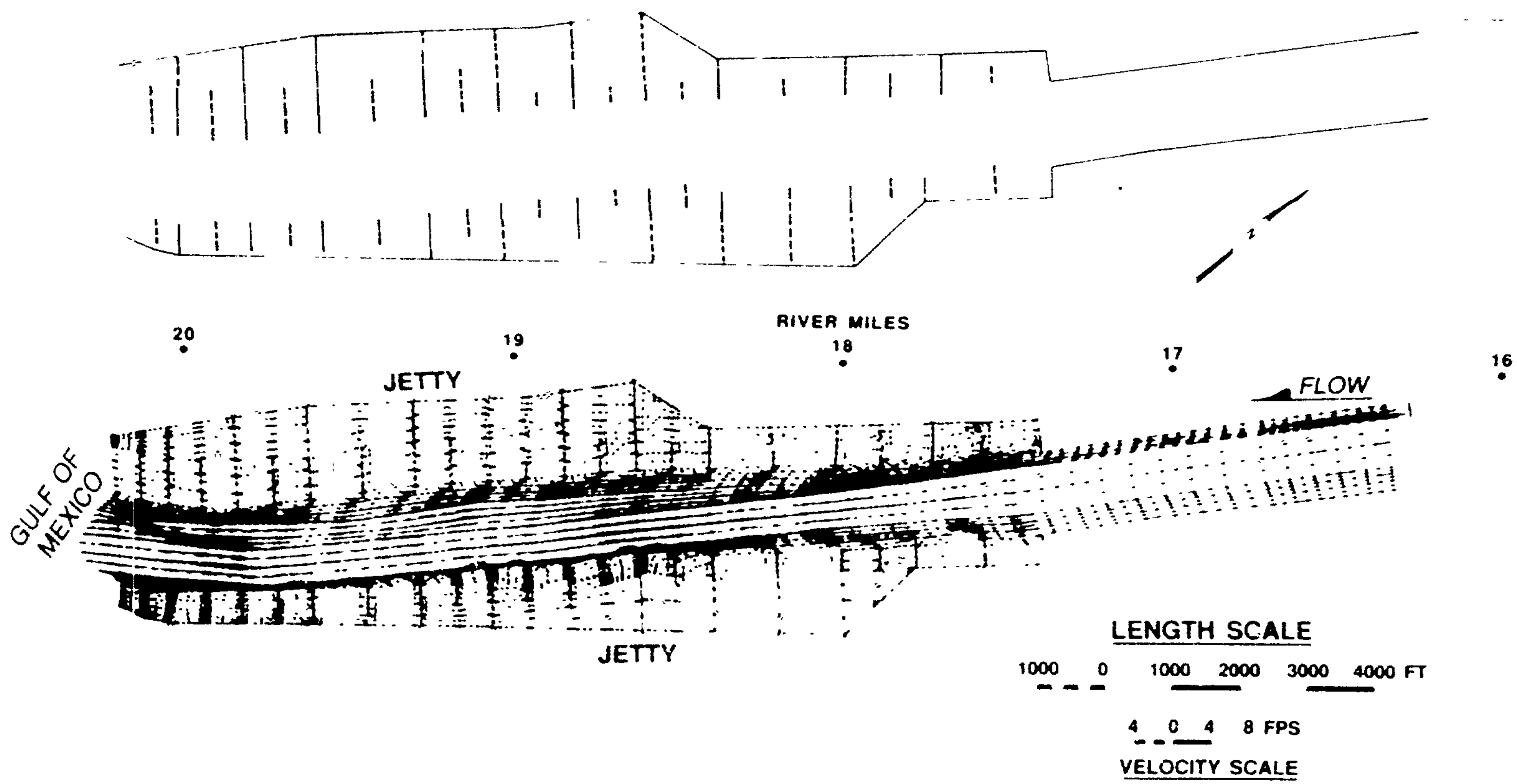


Figure 3. Plan C dike layout and velocity field

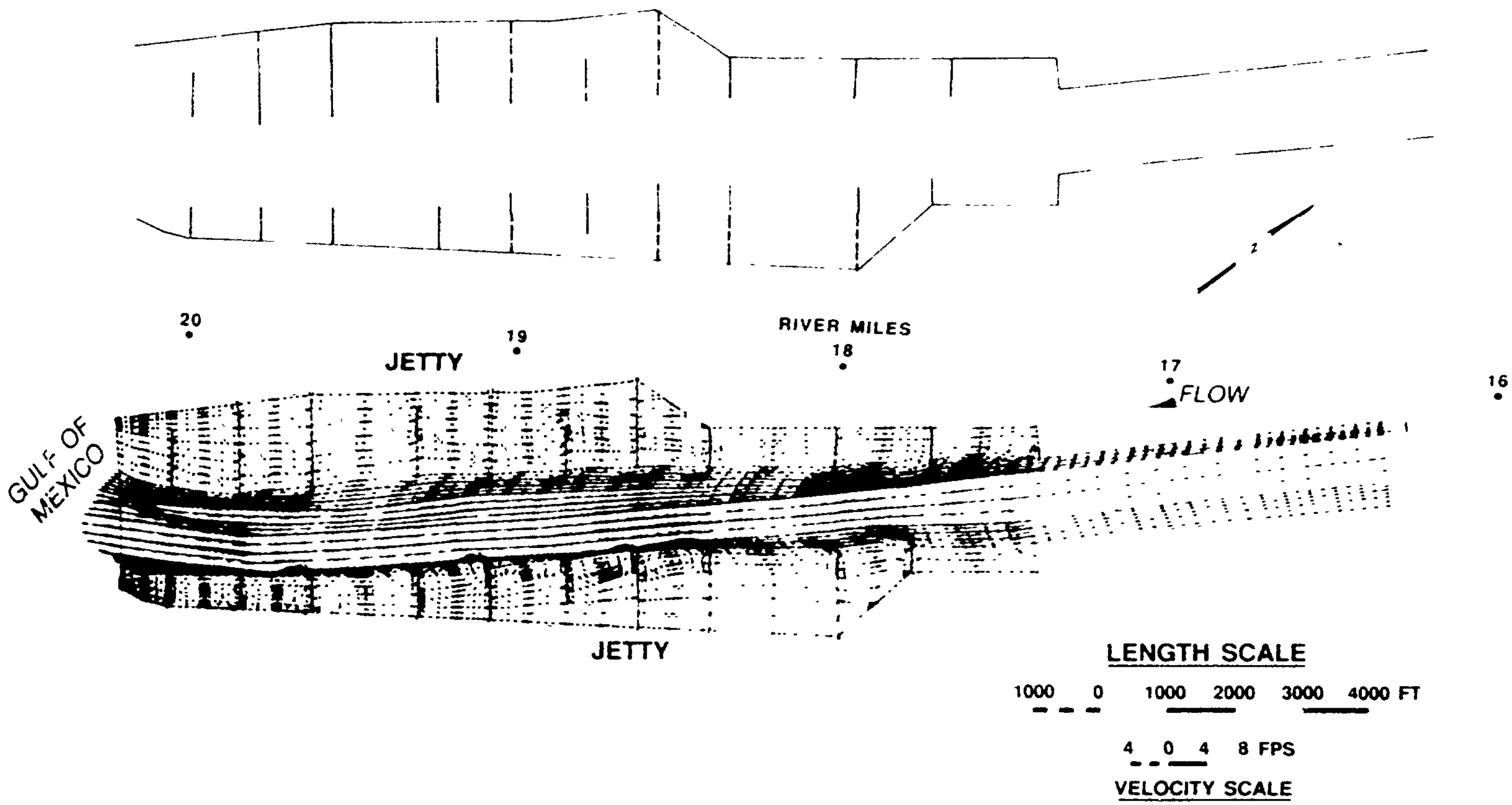


Figure 4. Plan E dike layout and velocity field

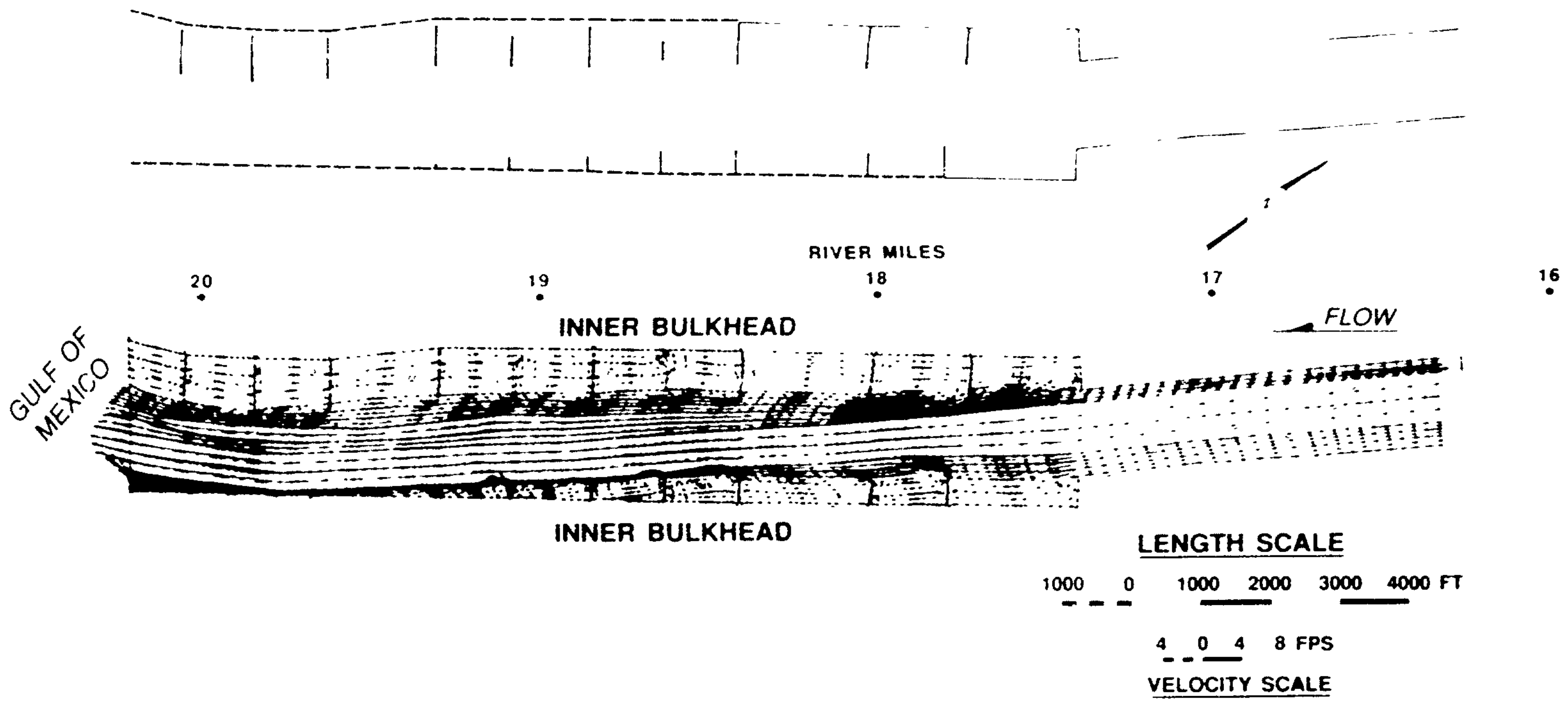


Figure 5. Plan G dike layout and velocity field

at the same time provided the tools to quickly look at a range of solutions instead of a restricted few.

Benefits of Project Performance. The estimated costs of the recommended plan, the bulkhead replacement, which was presented in the Corps of Engineers' 1984 Design Memorandum, is $47 million. Through the use of stone dikes, with shell core on geotextile fabric, as a substitute for 2.3 miles of the Raymond Concrete Pile Bulkhead, a savings of $25.4 million was realized in the shallow water reach. Through the use of advanced computer techniques, the project cost was reduced another $15 million in the remaining 1.7 mile reach as a result of the cost of construction of Plan E, the cheapest alternative presented in this paper.

Hindsight Observations. The recommended plan which was presented in the General Design Memorandum was engineered before the TABS-2 modeling system had advanced to the level of sophistication available today and was analyzed only with the HEC-2 Water Surface Profiles 1-D model. No suitable numerical tools existed at that time and physical models were not considered to be desirable because recent distorted-scale model tests conducted in 1980 had proven to be incapable of discerning small differences in velocity fields around lateral dikes. As a consequence, no effort was made during project formulation to find a suitable economical alternative. Instead, only a simple replacement structure was considered which proved to be excessively expensive. The study approach used with the TABS-2 modeling system on this project proved to be an appropriate tool to screen alternatives. The basic assumption that verification was not needed to provide adequate screening of the plans was essentially correct and provided additional cost-saving to the New Orleans District by elimination of a data collection program which would have been needed for verification of velocities and shoaling rates.

REFERENCES

S. B. Heltzel, W. D. Martin, R. C. Berger, D. R. Richards, Nov 1989. "An Analysis of Training Structure Designs in Southwest Pass, Mississippi River," Technical Report HL-89-22, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS

U.S. Army Corps of Engineers, Apr 1984. "General Design Memorandum Mississippi River, Baton Rouge to the Gulf of Mexico, LA, Southwest Pass and Bar Channel," Supplement No. 2, U.S. Army Engineer District, New Orleans, LA

U.S. Army Corps of Engineers, June 1990. "Riverside Review," Volume 2, Number 6, U.S. Army Engineer District, New Orleans, LA

An Analysis of Alternative Training Structures in Southwest Pass, Mississippi River

by

Cecil W. Soileau

SUMMARY OF DISCUSSION BY GARY R. DYHOUSE

Q: Are significant environmental problems encountered in the disposal of the dredge material?
A: No, the navigational channel is relatively small, compared to the dike field. We have been able to deposit the dredge material in the dike field for settlement and consolidation without significant adverse effects.

Q: Is a traffic control plan necessary in Southwest Pass?
A: Yes, especially during dredging periods.

Q: The TABS-2 process was handled by WES personnel. Do you have District people trained in TABS-2 now and, if so, what was the learning time?
A: This effort was also intended to get District personnel competent in TABS-2 and this has been accomplished. I would estimate that about one to one and a half man-years are needed to become well-versed in the program.

MUSKINGUM-CUNGE CHANNEL ROUTING

by

Gary W. Brunner[1]

I. INTRODUCTION

The Muskingum-Cunge channel routing technique is a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are: (1) the parameters of the model are physically based; (2) the method has been shown to compare well against the full unsteady flow equations over a wide range of flow situations (Ponce, 1983); and (3) the solution is independent of the user specified computation interval. The major limitations of the Muskingum-Cunge technique are that: (1) it cannot account for backwater effects; and (2) the method begins to diverge from the full unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections (i.e., channel slopes less than 1 ft./mile).

II. DEVELOPMENT OF EQUATIONS

The basic formulation of the equations is derived from the continuity equation and the diffusion form of the momentum equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_L \quad \text{(continuity)} \qquad (1)$$

$$S_f = S_o - \frac{\partial Y}{\partial x} \quad \text{(diffusion form of momentum equation)} \qquad (2)$$

By combining equations (1) and (2) and linearizing, the following convective diffusion equation is formulated (Miller and Cunge, 1975):

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = \mu \frac{\partial^2 Q}{\partial x^2} + cq_L \qquad (3)$$

Where:
Q = Discharge in cfs
A = Flow area in ft^2
t = Time in seconds
x = Distance along the channel in feet
Y = Depth of flow in feet
q_L = Lateral inflow per unit of channel length
S_f = Friction slope
S_o = Bed Slope
c = The wave celerity in the x direction as defined below.

[1] Hydraulic Engineer, Hydrologic Engineering Center

$$c = \frac{dQ}{dA}\bigg|_x \qquad (4)$$

The hydraulic diffusivity (μ) is expressed as follows:

$$\mu = \frac{Q}{2BS_o} \qquad (5)$$

where B is the top width of the water surface.

Following a Muskingum-type formulation, with lateral inflow, the continuity equation (1) is discretized on the x-t plane (Figure 1) to yield:

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n + C_4 Q_L \qquad (6)$$

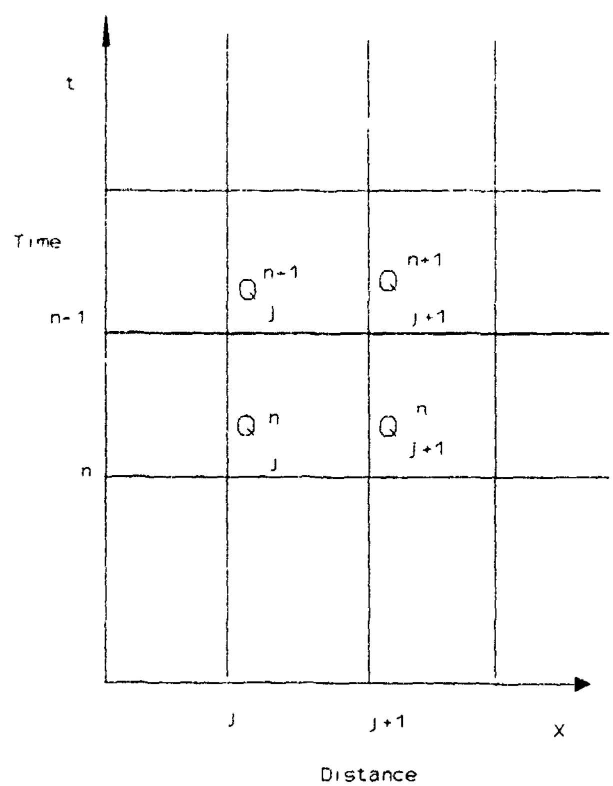


Figure 1: Discretization on x-t plane of the variable parameter Muskingum-Cunge Model.

It is assumed that the storage in the reach is expressed as the classical Muskingum storage:

$$S = K\,[X\,I + (1 - X)\,O] \qquad (7)$$

Where: S = channel storage
K = cell travel time (seconds)
X = weighting factor
I = inflow
O = outflow

Therefore, the coefficients can be expressed as follows:

$$C_1 = \frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2\,(1-X)} \qquad C_2 = \frac{\frac{\Delta t}{K} - 2X}{\frac{\Delta t}{K} + 2\,(1-X)}$$

$$C_3 = \frac{2(1-X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2\,(1-X)} \qquad C_4 = \frac{2(\frac{\Delta t}{K})}{\frac{\Delta t}{K} + 2\,(1-X)}$$

$$Q_L = q_L \Delta x$$

In the Muskingum equation the amount of diffusion is based on the value of X, which varies between 0.0 and 0.5. The Muskingum X parameter is not directly related to physical channel properties. The diffusion obtained with the Muskingum technique is a function of how the equation is solved, and is therefore considered numerical diffusion rather than physical. In the Muskingum-Cunge formulation, the amount of diffusion is controlled by forcing the numerical diffusion to match the physical diffusion (μ) from equations (3) and (5). The Muskingum-Cunge equation is therefore considered an approximation of the convective diffusion equation (3). As a result, the parameters K and X are expressed as follows (Cunge, 1969 and Ponce, 1978):

$$K = \frac{\Delta x}{c} \qquad \text{. . . . . (8)}$$

$$X = \tfrac{1}{2}\,(1 - \frac{Q}{BS_o c \Delta x}) \qquad \text{. . . . . (9)}$$

Then, the Courant (C) and cell Reynolds (D) numbers can be defined as:

$$C = c\,\frac{\Delta t}{\Delta x} \qquad \text{. . . . . (10)}$$

and

$$D = \frac{Q}{BS_o c \Delta x} \qquad \text{. . . . . (11)}$$

The routing coefficients for the non-linear diffusion method (Muskingum-Cunge) are then expressed as follows:

$$C_1 = \frac{1+C-D}{1+C+D}$$

$$C_2 = \frac{-1+C+D}{1+C+D}$$

$$C_3 = \frac{1-C+D}{1+C+D}$$

$$C_4 = \frac{2C}{1+C+D}$$

in which the dimensionless numbers C and D are expressed in terms of physical quantities (Q, B, S_o, and c) and the grid dimensions (Δx and Δt).

III. SOLUTION OF THE EQUATIONS

The method is non-linear in that the flow hydraulics (Q, B, c), and therefore the routing coefficients (C_1, C_2, C_3, and C_4) are re-calculated for every Δx distance step and Δt time step. An iterative four-point averaging scheme is used to solve for c, B and Q. This process has been described in detail by Ponce (1986).

Values for Δt and Δx are chosen internally by the model for accuracy and stability. First, Δt is evaluated by looking at the following 3 criteria and selecting the smallest value:

1. The user defined computation interval, NMIN, from the first field of the IT record.

2. The time of rise of the inflow hydrograph divided by 20 ($^{Tr}/_{20}$).

3. The travel time of the channel reach.

Once Δt is chosen, Δx is defined as follows:

$$\Delta x = c\Delta t \qquad \text{. . . . . (12)}$$

but Δx must also meet the following criteria to preserve consistency in the method (Ponce, 1983):

$$\Delta x < \tfrac{1}{2}\,(c\Delta t + \frac{Q_o}{BS_oC}) \qquad \text{. . . . . (13)}$$

where Q_o is the reference flow and Q_B is the baseflow taken from the inflow hydrograph as:

$$Q_o = Q_B + 0.50\ (Q_{peak} - Q_B)$$

Δx is chosen as the smaller value from the two criteria. The values chosen by the program for Δx and Δt are printed in the output, along with the computed peak flow. Before the hydrograph is used in subsequent operations, or printed in the hydrograph tables, it is converted back to the user-specified computation interval. The user should always check to see if the interpolation back to the user-specified computation interval has reduced the peak flow significantly. If the peak flow computed from the internal computation interval is markedly greater than the hydrograph interpolated back to the user-specified computation interval, the user-specified computation interval should be reduced and the model should be executed again.

IV. DATA REQUIREMENTS

Data for the Muskingum-Cunge method consist of the following:

1. Representative channel cross section.

2. Reach length, L.

3. Manning roughness coefficients, n (for main channel and overbanks).

4. Channel bed slope, S_0.

The method can be used with a simple cross section (i.e., trapezoid, rectangle, square, triangle, or circular pipe), or a more detailed 8-point cross section can be provided. If one of the simple channel configurations is used, Muskingum-Cunge routing can be accomplished through the use of a single RD record as follows:

KK . . . . Station Computation Identifier
RD . . . . Muskingum-Cunge Data

If the more detailed 8-point cross section is used, enter the following sequence of records:

KK . . . . Station Computation Identifier
RD . . . . Blank record to indicate Muskingum - Cunge routing
RC . . . . 8-point Cross Section Hydraulic Data
RX . . . . 8-point Cross-Section Station Data
RY . . . . 8-point Cross-Section Elevation Data

When using the 8-point cross section, it is not necessary to fill out the data for the RD record. All of the necessary information is taken from the RC, RX and RY records.

V. INPUT AND OUTPUT EXAMPLE

The use of Muskingum-Cunge channel routing is demonstrated here in the development of a rainfall-runoff model for Kempton Creek. The watershed has been subdivided into three separate catchments, as shown in Figure 2. Clark's unit hydrograph and the SCS Curve Number method were used to evaluate local runoff from each of the subbasins. Channel routing from control point CP1 to CP2 and from CP2 to CP3 was accomplished with Muskingum-Cunge routing.

KEMPTON CREEK WATERSHED

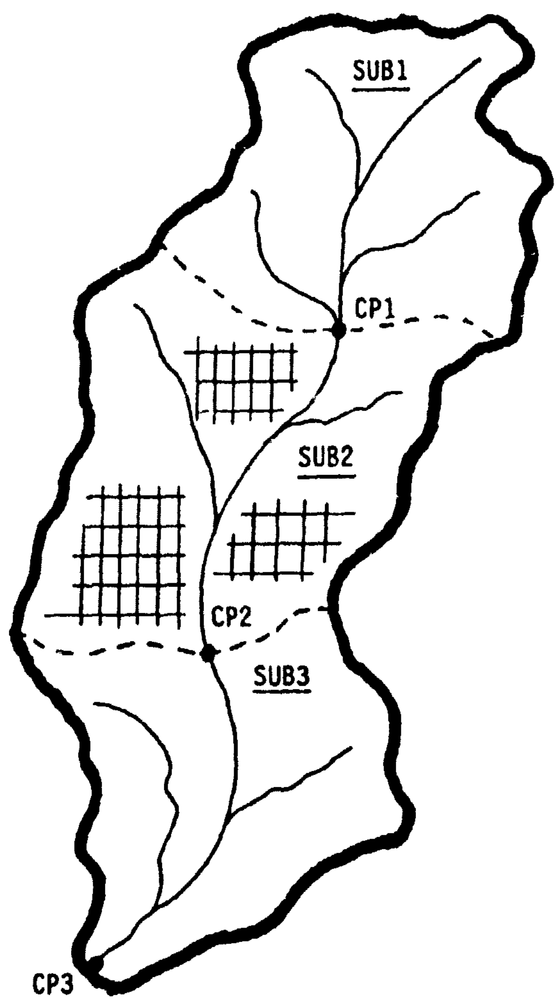


Figure 2. Kempton Creek Watershed for Muskingum-Cunge channel routing example.

Subbasin 2 (SUB2) is heavily urbanized with commercial and residential land use. The channel from CP1 to CP2 is a concrete lined trapezoidal channel with the following dimensions:

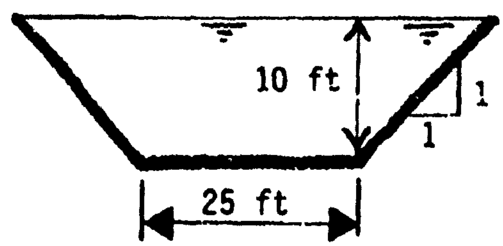


Figure 3. Trapezoidal channel.

Both subbasins 1 and 3 are completely undeveloped. The channel between CP2 and CP3 is in its natural state. A representative 8-point cross section has been fit to match the main channel and overbank flows through the reach as shown below:

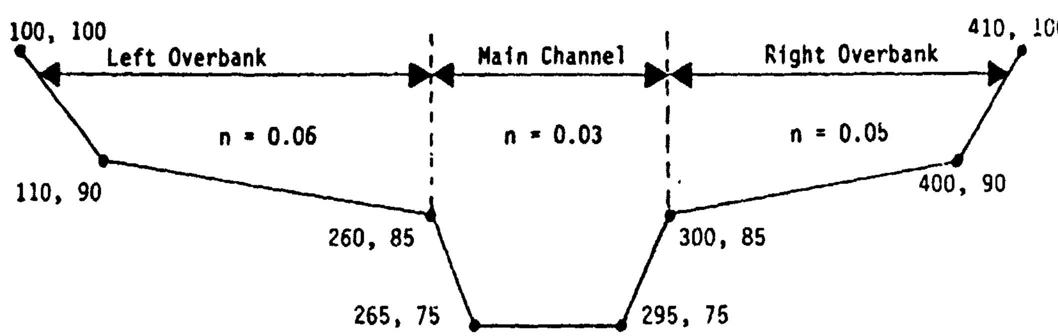


Figure 4. 8-point Cross Section

Listings of the required input data and the resulting output are shown in table 1. For the channel routing from CP1 to CP2, it is only necessary to have an RD record. Use of the RD record by itself means that the channel geometry can be described with a simple geometric element, such as a trapezoid. For the routing reach between CP2 and CP3, it is necessary to also include RC, RX, and RY records to describe the geometry through this reach. When using the 8-point cross-section option, the RD record only serves to indicate a Muskingum-Cunge channel routing is being performed. All of the necessary information is obtained from the RC, RX, and RY records.

TABLE 1

Example Problem : Input and Output

```
                              HEC-1 INPUT                                                  PAGE  1

LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1          ID  TEST EXAMPLE NO. 15.  MUSKINGUM-CUNGE CHANNEL ROUTING EXAMPLE
   2          ID  GARY W. BRUNNER  APRIL 18, 1989
   3          IT      15 18APR89    1100      60
   4          IO       5
              *

   5          KK   SUB1
   6          KM   RUNOFF CALCULATION FOR SUB1
   7          BA   25.0
   8          PB    3.5
   9          PI    0.2     0.3     0.5     0.8     1.0     0.8     0.6     0.4     0.2     0.1
  10          BF   -1.0    -.05    1.02
  11          LS    0.5      65
  12          UC    3.5     3.0
              *

  13          KK  ROUT1
  14          KM  ROUTE SUB1 HYDROGRAPH FROM CP1 TO CP2
  15          KO       1
  16          RD  31680  0.0008   0.015           TRAP      25     1.0
              *

  17          KK   SUB2
  18          KM   LOCAL RUNOFF FROM SUBBASIN SUB2
  19          BA   35.0
  20          PB    3.0
  21          LS    0.5      75      35
  22          UC    2.8     2.1
              *

  23          KK   SUB2
  24          KM   COMBINE LOCAL SUB2 AND ROUTED SUB1 HYDROGRAPHS
  25          HC       2
              *

  26          KK  ROUT2
  27          KM  ROUTE TOTAL FLOW AT SUB2 FROM CP2 TO CP3
  28          KO       1
  29          RD
  30          RC   0.06    0.03    0.05   29040  0.0007      96
  31          RX    100     110     260     265     295     300     400     410
  32          RY    100      90      85      75      75      85      90     100
              *

  33          KK   SUB3
  34          KM   LOCAL RUNOFF FROM SUBBASIN SUB3
  35          BA   32.5
  36          PB    2.9
  37          LS    0.5      70
  38          UC    4.0     3.5
              *

  39          KK   SUB3
  40          KM   COMBINE LOCAL SUB3 WITH ROUTED FROM SUB2
  41          HC       2
              *
  42          ZZ
```

```
*****************************************                                          *****************************************
*                                       *                                          *                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                          *     U.S. ARMY CORPS OF ENGINEERS      *
*             FEBRUARY 1981             *                                          *  THE HYDROLOGIC ENGINEERING CENTER    *
*           REVISED 05 DEC 88           *                                          *          609 SECOND STREET            *
*                                       *                                          *       DAVIS, CALIFORNIA 95616         *
*  RUN DATE  05/01/1989  TIME  13:12:37 *                                          *          (916) 551-1748               *
*                                       *                                          *                                       *
*****************************************                                          *****************************************


                            TEST EXAMPLE NO. 15.  MUSKINGUM-CUNGE CHANNEL ROUTING EXAMPLE
                            GARY W. BRUNNER  APRIL 18, 1989

    4 IO         OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

      IT         HYDROGRAPH TIME DATA
                        NMIN          15  MINUTES IN COMPUTATION INTERVAL
                       IDATE     18APR89  STARTING DATE
                       ITIME        1100  STARTING TIME
                          NQ          60  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE     19APR89  ENDING DATE
                      NDTIME        0145  ENDING TIME
                       ICENT          19  CENTURY MARK

                   COMPUTATION INTERVAL     .25 HOURS
                        TOTAL TIME BASE   14.75 HOURS

          ENGLISH UNITS
               DRAINAGE AREA         SQUARE MILES
               PRECIPITATION DEPTH   INCHES
               LENGTH, ELEVATION     FEET
               FLOW                  CUBIC FEET PER SECOND
               STORAGE VOLUME        ACRE-FEET
               SURFACE AREA          ACRES
               TEMPERATURE           DEGREES FAHRENHEIT

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***
```

```
              **************
              *            *
13 KK         *   ROUT1    *
              *            *
              **************

15 KO           OUTPUT CONTROL VARIABLES
                      IPRNT          1  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

              HYDROGRAPH ROUTING DATA

16 RD           MUSKINGUM-CUNGE CHANNEL ROUTING
                          L     31680.  CHANNEL LENGTH
                          S      .0008  SLOPE
                          N       .015  CHANNEL ROUGHNESS COEFFICIENT
                         CA        .00  CONTRIBUTING AREA
                      SHAPE       TRAP  CHANNEL SHAPE
                         WD      25.00  BOTTOM WIDTH OR DIAMETER
                          Z       1.00  SIDE SLOPE
```

***

COMPUTED MUSKINGUM-CUNGE PARAMETERS

COMPUTATION TIME STEP

ELEMENT	ALPHA	M	DT (MIN)	DX (FT)	PEAK (CFS)	TIME TO PEAK (MIN)	VOLUME (IN)	MAXIMUM CELERITY (FPS)
MAIN	.42	1.56	12.00	6336.00	3330.12	300.00	1.05	12.85

INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

MAIN	.42	1.56	15.00		3330.12	300.00	1.05	

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1425E+04 EXCESS= .0000E+00 OUTFLOW= .1405E+04 BASIN STORAGE= .2808E+02 PERCENT ERROR= -.5

****

HYDROGRAPH AT STATION ROUT1

****

DA MON HRMN	ORD	FLOW	*	DA MON HRMN	ORD	FLOW	*	DA MON HRMN	ORD	FLOW	*	DA MON HRMN	ORD	FLOW
18 APR 1100	1	25.	*	18 APR 1445	16	2064.	*	18 APR 1830	31	1858.	*	18 APR 2215	46	607.
18 APR 1115	2	25.	*	18 APR 1500	17	2481.	*	18 APR 1845	32	1721.	*	18 APR 2230	47	565.
18 APR 1130	3	25.	*	18 APR 1515	18	2818.	*	18 APR 1900	33	1594.	*	18 APR 2245	48	526.
18 APR 1145	4	25.	*	18 APR 1530	19	3077.	*	18 APR 1915	34	1479.	*	18 APR 2300	49	489.
18 APR 1200	5	25.	*	18 APR 1545	20	3248.	*	18 APR 1930	35	1371.	*	18 APR 2315	50	456.
18 APR 1215	6	25.	*	18 APR 1600	21	3330.	*	18 APR 1945	36	1271.	*	18 APR 2330	51	425.
18 APR 1230	7	25.	*	18 APR 1615	22	3315.	*	18 APR 2000	37	1179.	*	18 APR 2345	52	396.
18 APR 1245	8	25.	*	18 APR 1630	23	3228.	*	18 APR 2015	38	1094.	*	19 APR 0000	53	369.
18 APR 1300	9	27.	*	18 APR 1645	24	3084.	*	18 APR 2030	39	1016.	*	19 APR 0015	54	344.
18 APR 1315	10	39.	*	18 APR 1700	25	2907.	*	18 APR 2045	40	943.	*	19 APR 0030	55	321.
18 APR 1330	11	78.	*	18 APR 1715	26	2714.	*	18 APR 2100	41	875.	*	19 APR 0045	56	300.
18 APR 1345	12	183.	*	18 APR 1730	27	2522.	*	18 APR 2115	42	813.	*	19 APR 0100	57	280.
18 APR 1400	13	533.	*	18 APR 1745	28	2337.	*	18 APR 2130	43	756.	*	19 APR 0115	58	262.
18 APR 1415	14	1067	*	18 APR 1800	29	2164.	*	18 APR 2145	44	702.	*	19 APR 0130	59	243.
18 APR 1430	15	1589.	*	18 APR 1815	30	2005.	*	18 APR 2200	45	652.	*	19 APR 0145	60	236.

****

PEAK FLOW (CFS)	TIME (HR)		MAXIMUM AVERAGE FLOW 6-HR	24-HR	72-HR	14.75-HR
3330.	5.00	(CFS)	2268.	1153.	1153.	1153.
		(INCHES)	.844	1.054	1.054	1.054
		(AC-FT)	1125.	1405.	1405.	1405.

CUMULATIVE AREA = 25.00 SQ MI

```
                ***************
                *             *
    26 KK       *    ROUT2    *
                *             *
                ***************

    28 KO          OUTPUT CONTROL VARIABLES
                         IPRNT           1  PRINT CONTROL
                         IPLOT           2  PLOT CONTROL
                         QSCAL          0.  HYDROGRAPH PLOT SCALE

                 HYDROGRAPH ROUTING DATA

    29 RD          MUSKINGUM-CUNGE CHANNEL ROUTING

    30 RC          NORMAL DEPTH CHANNEL
                          ANL        .060  LEFT OVERBANK N-VALUE
                         ANCH        .030  MAIN CHANNEL N-VALUE
                          ANR        .050  RIGHT OVERBANK N-VALUE
                        RLNTH      29040.  REACH LENGTH
                          SEL       .0007  ENERGY SLOPE
                        ELMAX        96.0  MAX. ELEV. FOR STORAGE/OUTFLOW CALCULATION

                                          CROSS-SECTION DATA
                           --- LEFT OVERBANK --- + ------ MAIN CHANNEL ------- + --- RIGHT OVERBANK ---
    32 RY        ELEVATION    100.00    90.00    85.00    75.00    75.00    85.00    90.00   100.00
    31 RX        DISTANCE     100.00   110.00   260.00   265.00   295.00   300.00   400.00   410.00

                                                       ***

                                      COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

         STORAGE        .00    22.51    45.84    69.98    94.94   120.71   147.29   174.69   202.90   231.93
         OUTFLOW        .00    45.55   141.88   274.39   437.13   626.66   840.79  1078.04  1337.39  1618.12
        ELEVATION     75.00    76.11    77.21    78.32    79.42    80.53    81.63    82.74    83.84    84.95

         STORAGE     279.87   368.49   497.82   667.88   875.06  1090.26  1307.08  1525.54  1745.62  1967.33
         OUTFLOW    1984.99  2443.41  3029.47  3774.70  4754.58  5986.67  7389.58  8951.15 10662.29 12515.79
        ELEVATION     86.05    87.16    88.26    89.37    90.47    91.58    92.68    93.79    94.89    96.00


                                 COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                     COMPUTATION TIME STEP
              ELEMENT    ALPHA      M        DT        DX       PEAK    TIME TO    VOLUME    MAXIMUM
                                                                          PEAK               CELERITY
                                           (MIN)      (FT)     (CFS)    (MIN)      (IN)      (FPS)

               MAIN                        12.00   2904.00  10016.39   348.00      1.44       3.86


                                    INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL


               MAIN                        15.00            10008.47   360.00      1.44

CONTINUITY SUMMARY (AC-FT) - INFLOW= .4771E+04 EXCESS= .0000E+00 OUTFLOW= .4618E+04 BASIN STORAGE= .1125E+03 PERCENT ERROR=    .9
```

HYDROGRAPH AT STATION ROUT2

DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW
18 APR 1100	1	60.	18 APR 1445	16	3090.	18 APR 1830	31	7877.	18 APR 2215	46	2298.
18 APR 1115	2	60.	18 APR 1500	17	3853.	18 APR 1845	32	7431.	18 APR 2230	47	1860.
18 APR 1130	3	60.	18 APR 1515	18	4914.	18 APR 1900	33	7000.	18 APR 2245	48	1567.
18 APR 1145	4	60.	18 APR 1530	19	6160.	18 APR 1915	34	6592.	18 APR 2300	49	1382.
18 APR 1200	5	60.	18 APR 1545	20	7401.	18 APR 1930	35	6207.	18 APR 2315	50	1264.
18 APR 1215	6	60.	18 APR 1600	21	8477.	18 APR 1945	36	5846.	18 APR 2330	51	1179.
18 APR 1230	7	60.	18 APR 1615	22	9250.	18 APR 2000	37	5507.	18 APR 2345	52	1112.
18 APR 1245	8	60.	18 APR 1630	23	9745.	18 APR 2015	38	5187.	19 APR 0000	53	1058.
18 APR 1300	9	60.	18 APR 1645	24	9985.	18 APR 2030	39	4876.	19 APR 0015	54	1014.
18 APR 1315	10	130.	18 APR 1700	25	10008.	18 APR 2045	40	4569.	19 APR 0030	55	974.
18 APR 1330	11	512.	18 APR 1715	26	9841.	18 APR 2100	41	4259.	19 APR 0045	56	938.
18 APR 1345	12	1106.	18 APR 1730	27	9556.	18 APR 2115	42	3937.	19 APR 0100	57	905.
18 APR 1400	13	1664.	18 APR 1745	28	9191.	18 APR 2130	43	3592.	19 APR 0115	58	875.
18 APR 1415	14	2093.	18 APR 1800	29	8775.	18 APR 2145	44	3209.	19 APR 0130	59	847.
18 APR 1430	15	2529.	18 APR 1815	30	8329.	18 APR 2200	45	2777.	19 APR 0145	60	836.

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	14.75-HR
(CFS)	(HR)					
10008.	6.00	(CFS)	7366.	3791.	3791.	3791.
		(INCHES)	1.141	1.444	1.444	1.444
		(AC-FT)	3652.	4621.	4621.	4621.

CUMULATIVE AREA = 60.00 SQ MI

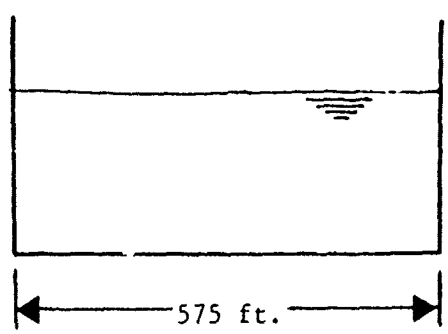
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	SUB1	3381.	4.50	2285.	1169.	1169.	25.00		
ROUTED TO	ROUT1	3330.	5.00	2268.	1153.	1153.	25.00		
HYDROGRAPH AT	SUB2	9862.	3.75	5816.	2763.	2763.	35.00		
2 COMBINED AT	CP2	12131.	4.00	7824.	3916.	3916.	60.00		
ROUTED TO	ROUT2	10008.	6.00	7366.	3791.	3791.	60.00		
HYDROGRAPH AT	SUB3	3091.	5.00	2225.	1202.	1202.	32.50		
2 COMBINED AT	CP3	12813.	5.75	9438.	4993.	4993.	92.50		

VI. COMPARISON WITH THE COMPLETE UNSTEADY FLOW EQUATIONS

In an effort to quantify the applicability and limitations of the Muskingum-Cunge routing technique, a comparison with the complete unsteady flow equations was undertaken. This analysis consisted of comparisons for prismatic channels of rectangular cross section, as well as more detailed compound cross sections (8 point cross sections) The analysis encompassed a wide range of channel slopes, varying from 42 ft/mi to 1 ft/mi. Rapidly rising hydrographs as well as slow rising hydrographs were routed through long channel sections with no lateral inflow. This analysis represents a very controlled routing situation, which is necessary to make a clear comparison between the variable coefficient Muskingum-Cunge method and the complete unsteady flow equations.

The first set of tests were for a rectangular channel with the following dimensions:



Channel Length = 95040 ft.

Manning's n = 0.03

Channel Slopes = 1 to 10 ft/mi.

Figure 5. Rectangular channel section with varying channel slopes.

Hydrographs were routed with the Muskingum-Cunge routing technique in HEC-1. The same channels and hydrographs were then analyzed with the National Weather Service DAMBRK model. This model was chosen as the standard for comparison because it has been nationally accepted and is considered one of the most accurate tools available for one dimensional channel flow. Extreme care was taken to ensure that the best possible answer was obtained with the DAMBRK model. Plots of the inflow and respective outflow hydrographs are shown in Figures 6 through 10. As shown in the plots, the Muskingum-Cunge method compares very well with the complete unsteady flow equations (DAMBRK model). The Muskingum-Cunge method begins to diverge from the DAMBRK answer when the channel slope is reduced to 1 ft/mi or less. The divergence is due to the fact that the inertial terms in the complete unsteady flow equations are becoming more dominant, compared to the bed slope, as the channel slope is decreased. The Muskingum-Cunge method does not account for the inertial effects, and consequently the method tends to show more diffusion than what may actually occur.

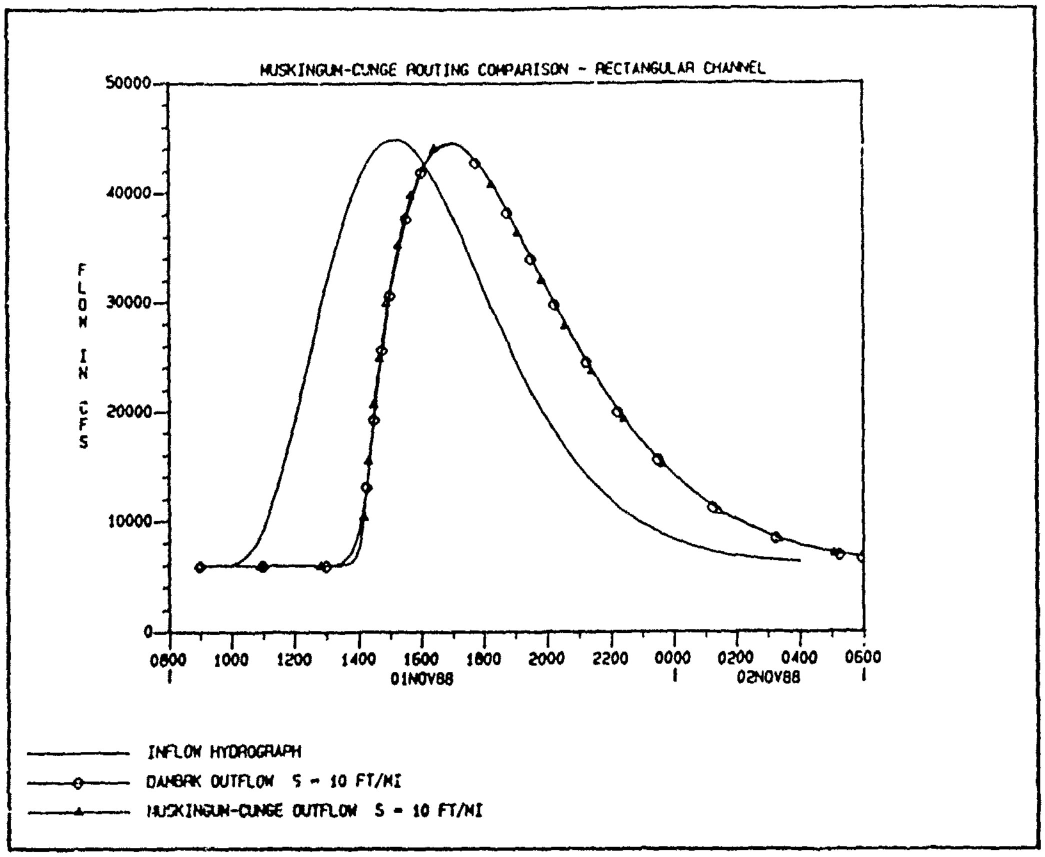


Figure 6. Rectangular channel with S=10 ft/mi (0.0019)

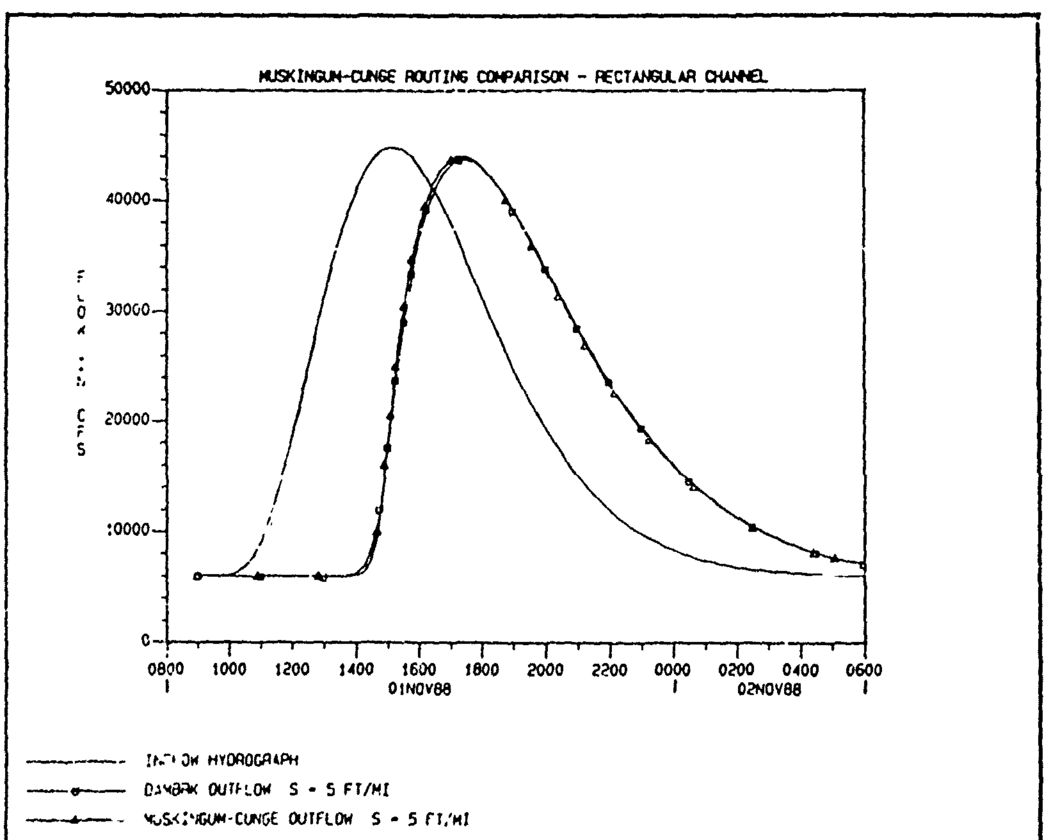


Figure 7. Rectangular channel with S=5 ft/mi (0.00095)

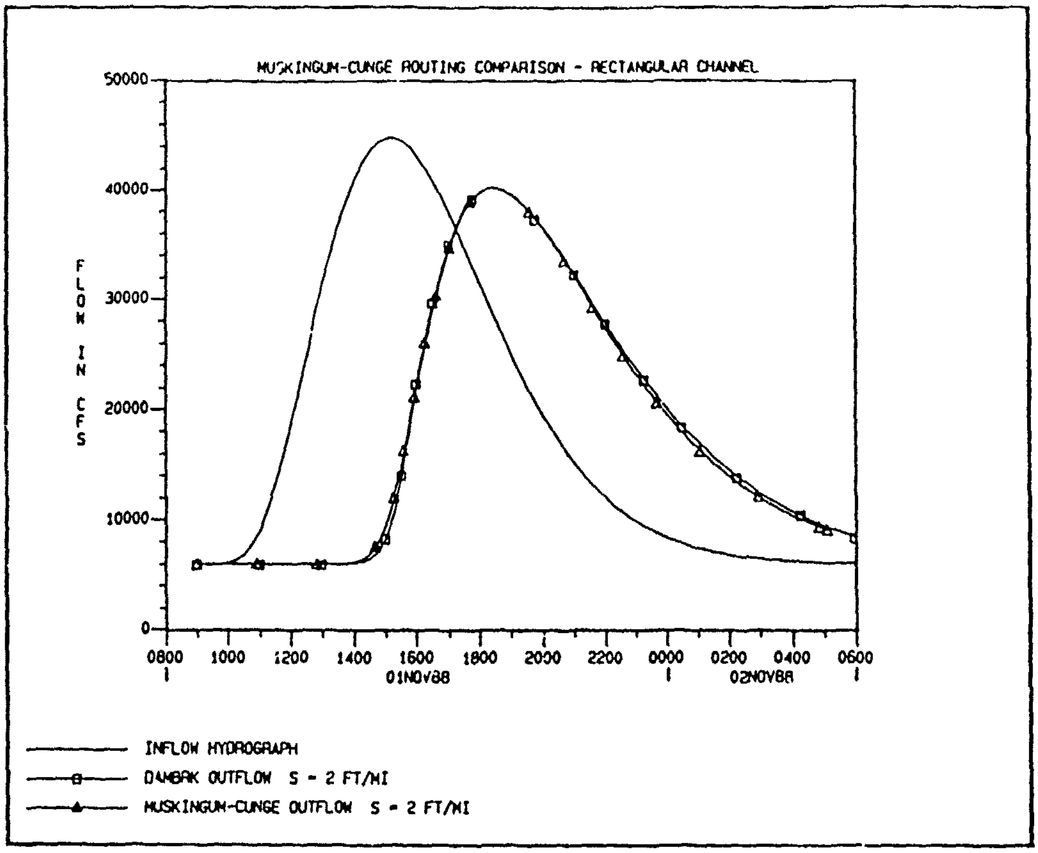


Figure 8. Rectangular channel with S=2 ft/mi (0.00038)

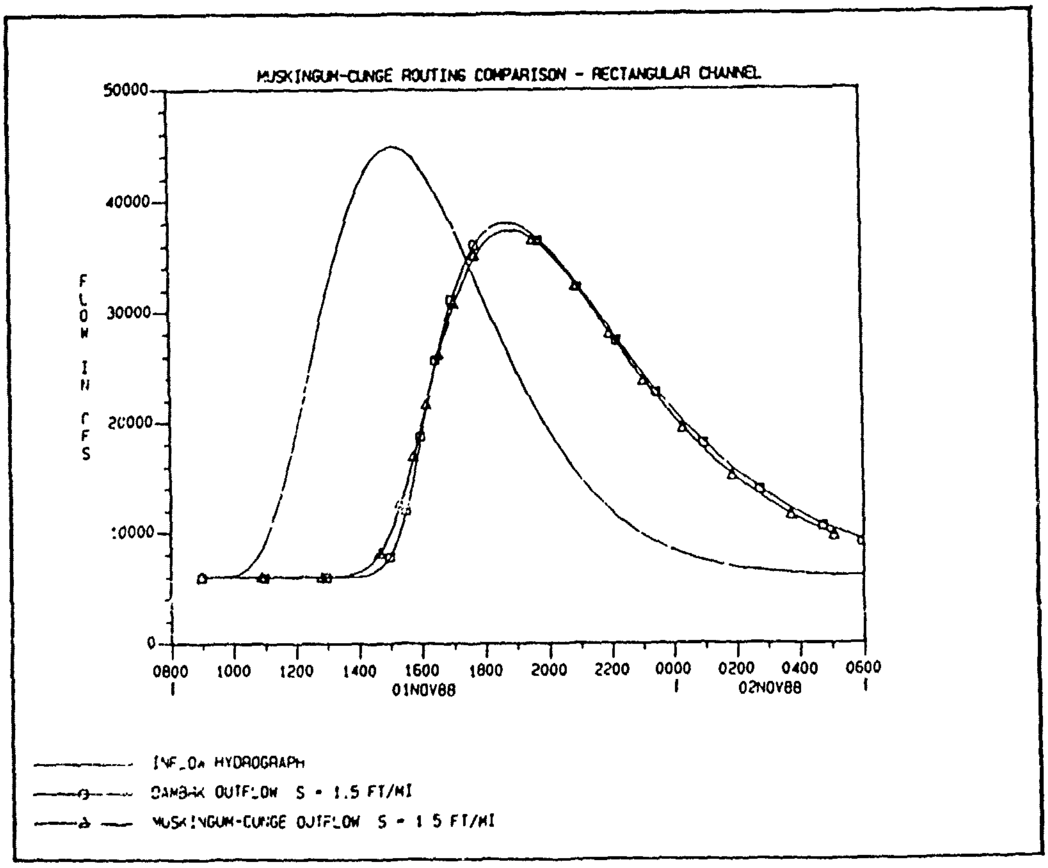


Figure 9. Rectangular channel with S=1.5 ft/mi (0.00028)

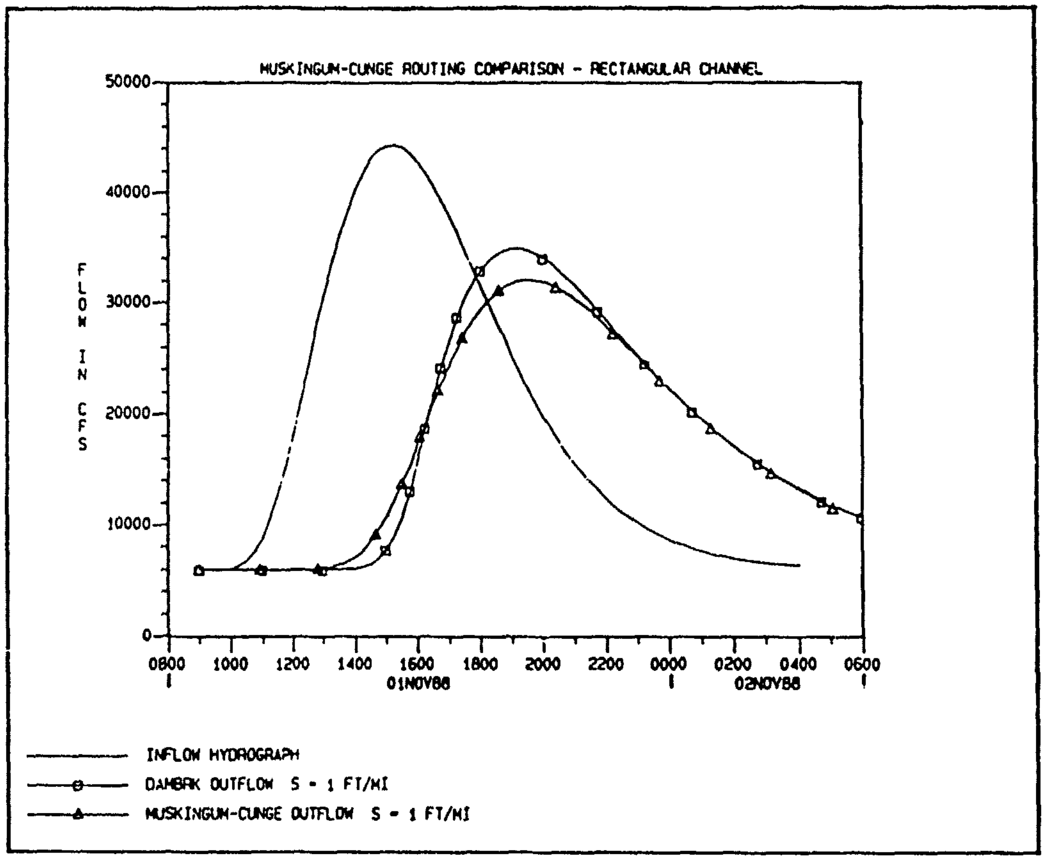


Figure 10. Rectangular channel with S=1 ft/mi (0.00019)

In the second series of tests, the effects of varying the rise time of the inflow hydrograph, as well as channel slope, were analyzed. In this analysis two different inflow hydrographs were used. The first inflow hydrograph has a time of rise of 45 minutes, peak flow of 70,622 cfs, and a time base of runoff equal to 2 hours. The second inflow hydrograph has a time of rise of 2 hours, peak flow of 70,622 cfs, and a time base of runoff equal to 6 hours. Channel slopes for this example were varied from 42 ft/mi to 1 ft/mi. The channel section was rectangular with the following hydrualic characteristics:

Channel length = 82,025 ft.

Manning's n = 0.04

Channel slopes = 1 to 42 ft/mi

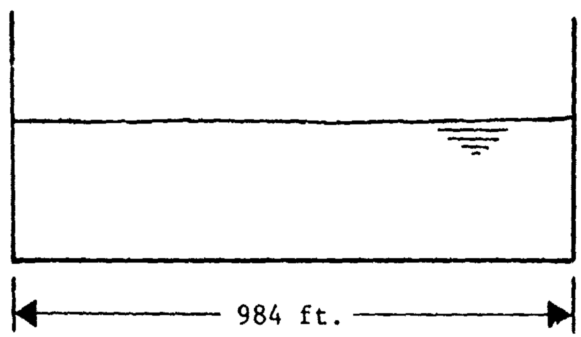


Figure 11. Rectangular channel with slopes from 42 ft/mi to 1 ft/mi.

Hydrographs were routed with the Muskingum-Cunge method as well as the NWS DAMBRK program. The resulting hydrographs are shown in figures 12 through 19. In general, the Muskingum-Cunge method compared very well for this series of tests. From review of the hydrograph plots, it is evident that the model performs better for slow rising hydrographs through steep channel sections. For rapidly rising hydrographs routed through flat river reaches, the Muskingum-Cunge method will tend to over predict the amount of diffusion. Although, the answers produced by the Muskingum-Cunge method may be within practical engineering limits. Also, these tests were performed for very long routing reaches with no lateral inflow, which is more of a dam breach type of analysis. For natural flood events, where lateral inflow will be added to the stream, the model will perform better over a wider range of channel slopes.

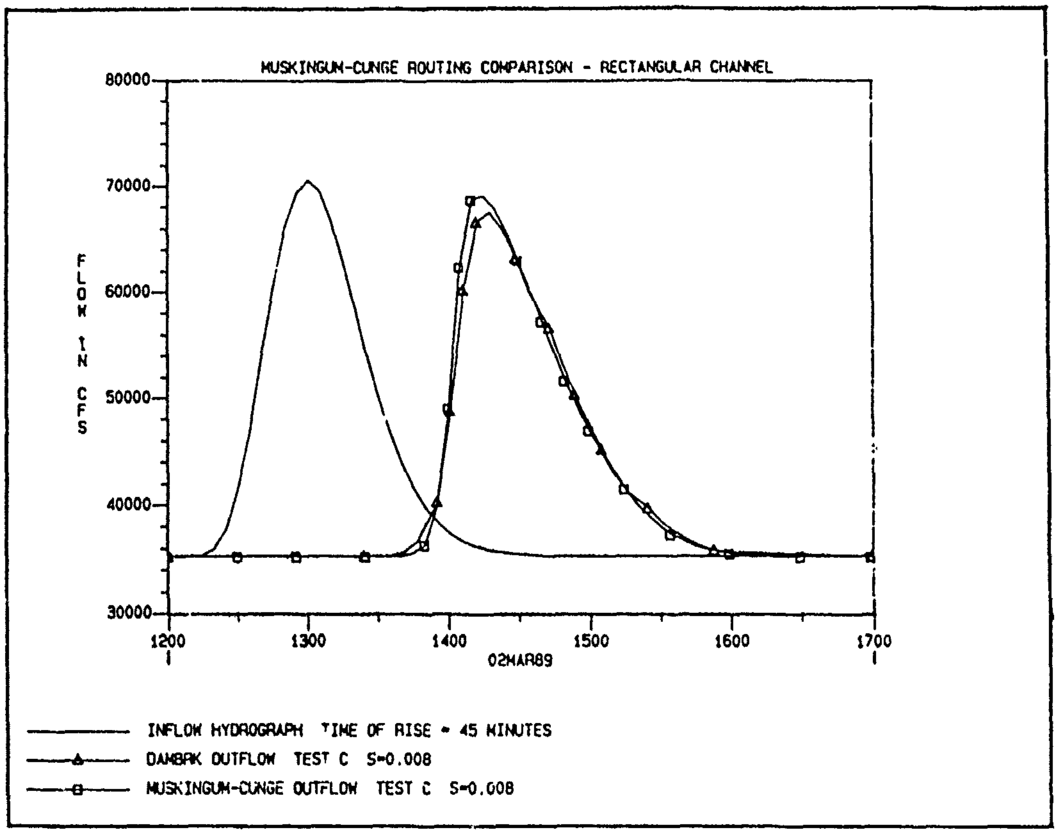


Figure 12. Rectangular channel, time of rise = 45 min, S = 0.008

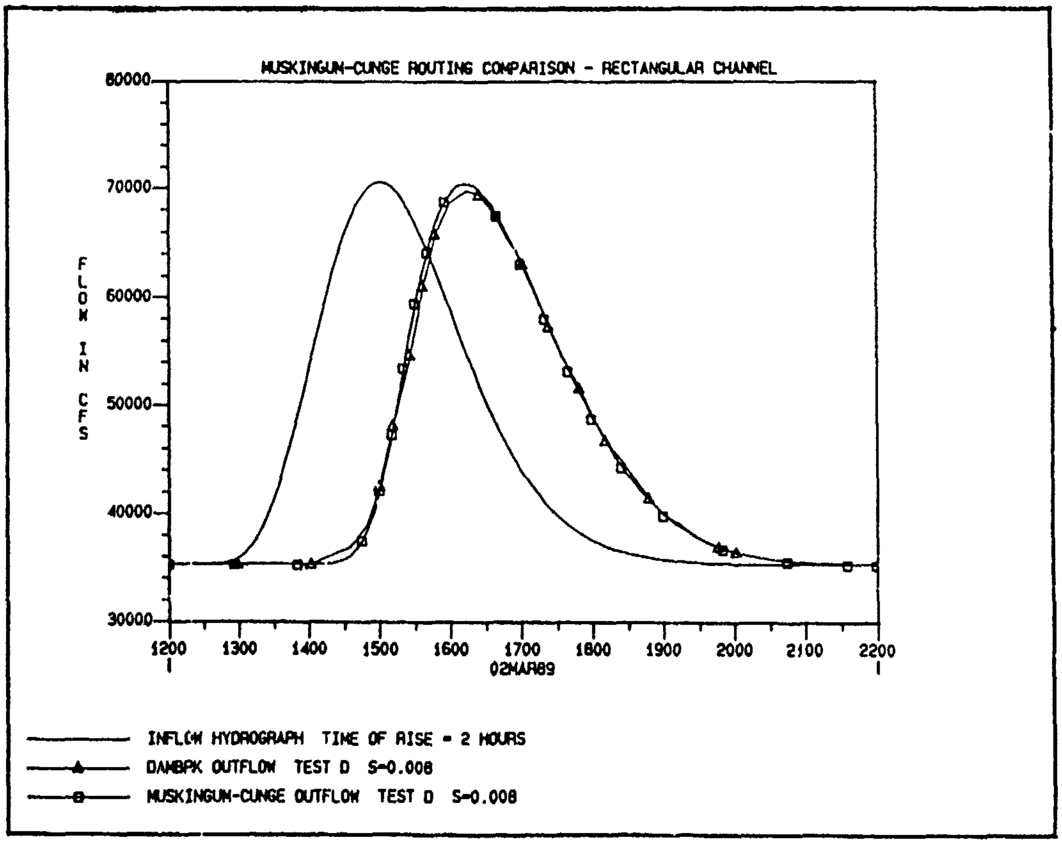


Figure 13. Rectangular channel, time of rise = 2 hrs, S = 0.008

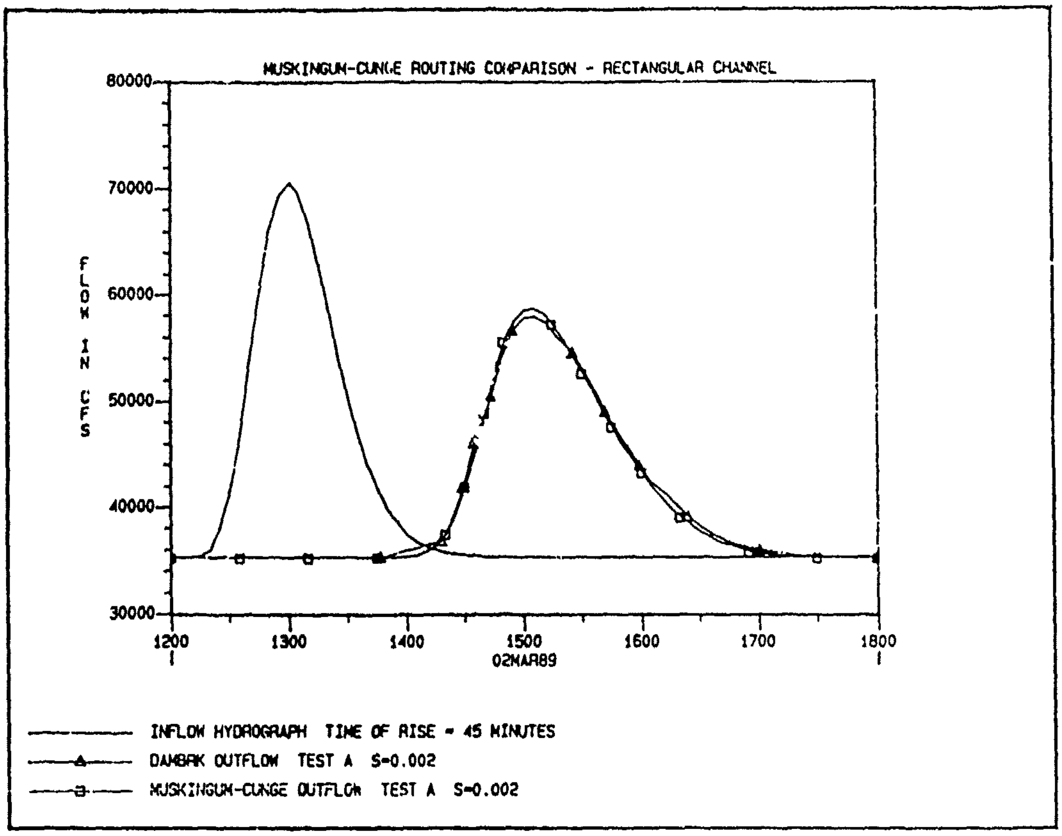


Figure 14. Rectangular channel, time of rise = 45 min, S = 0.002

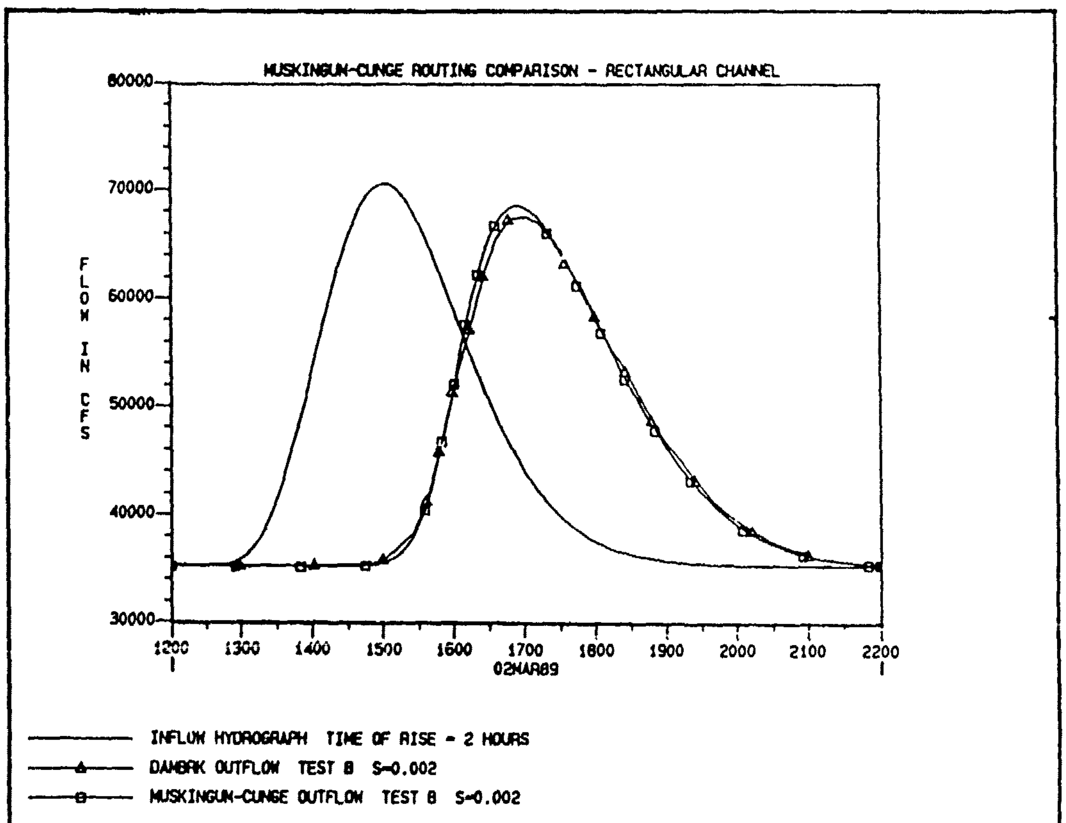


Figure 15. Rectangular channel, time of rise = 2 hrs, S = 0.002

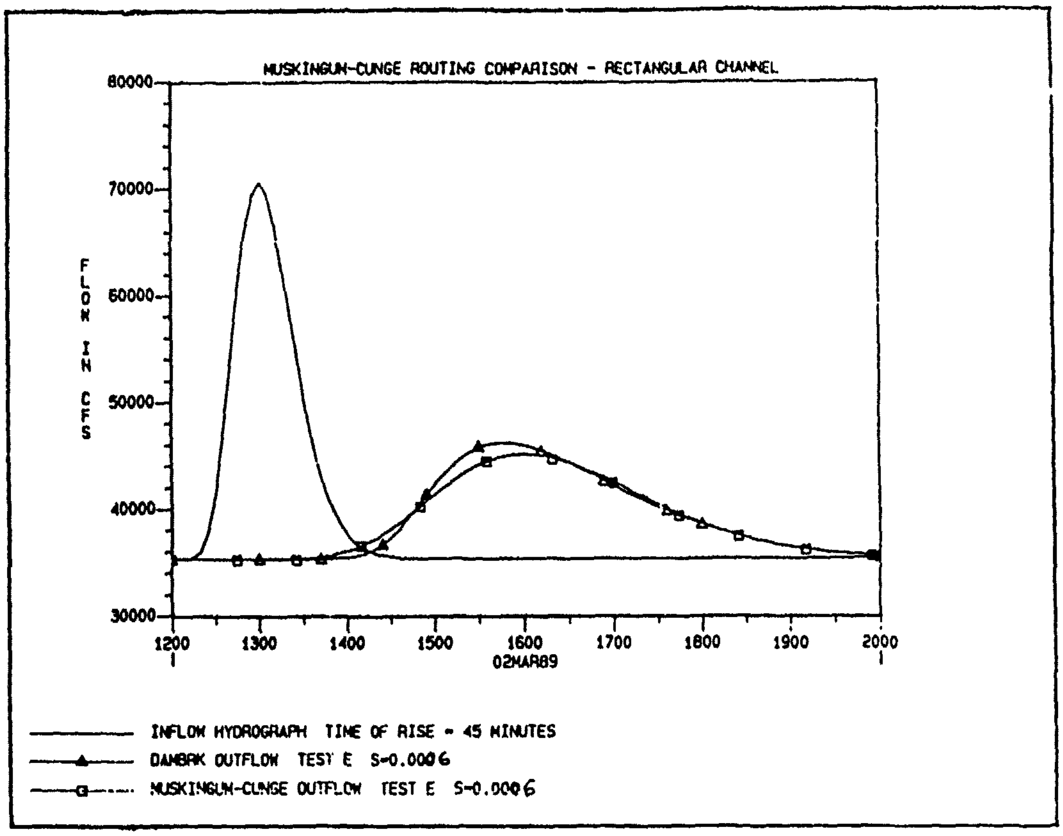


Figure 16. Rectangular channel, time of rise = 45 min, S = 0.0006

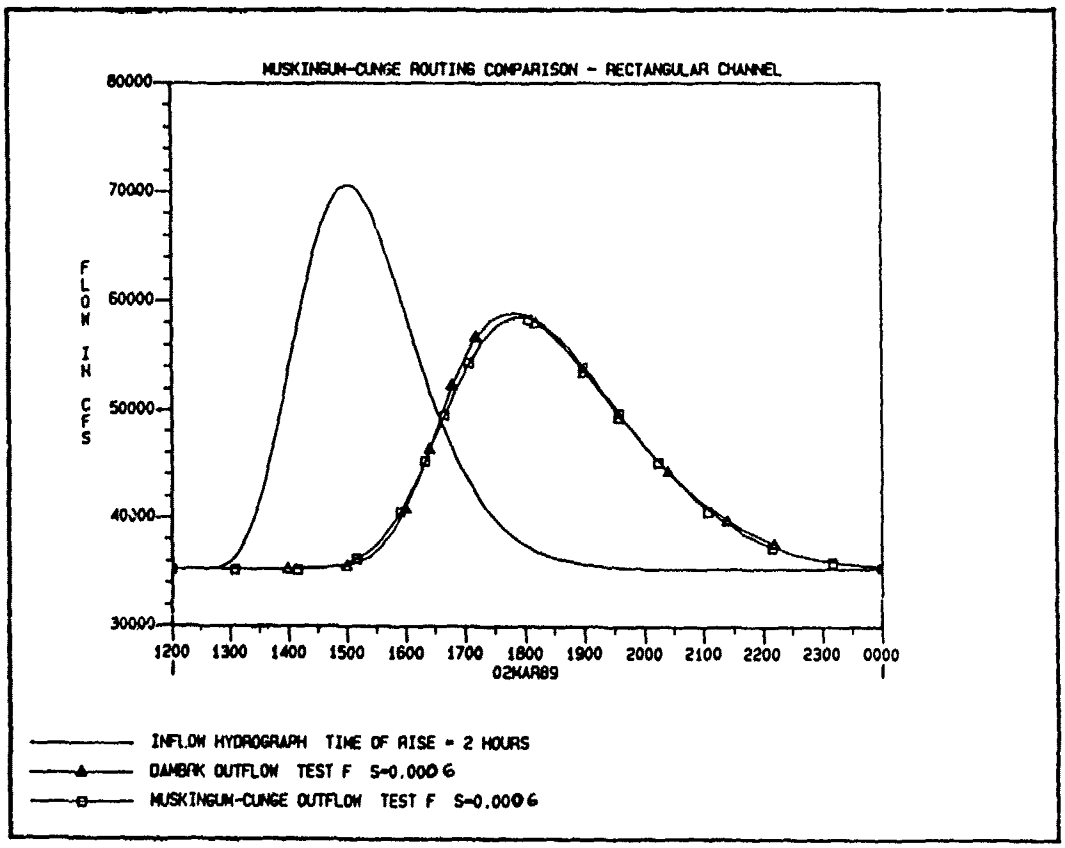


Figure 17. Rectangular channel, time of rise = 2 hrs, S = 0.0006

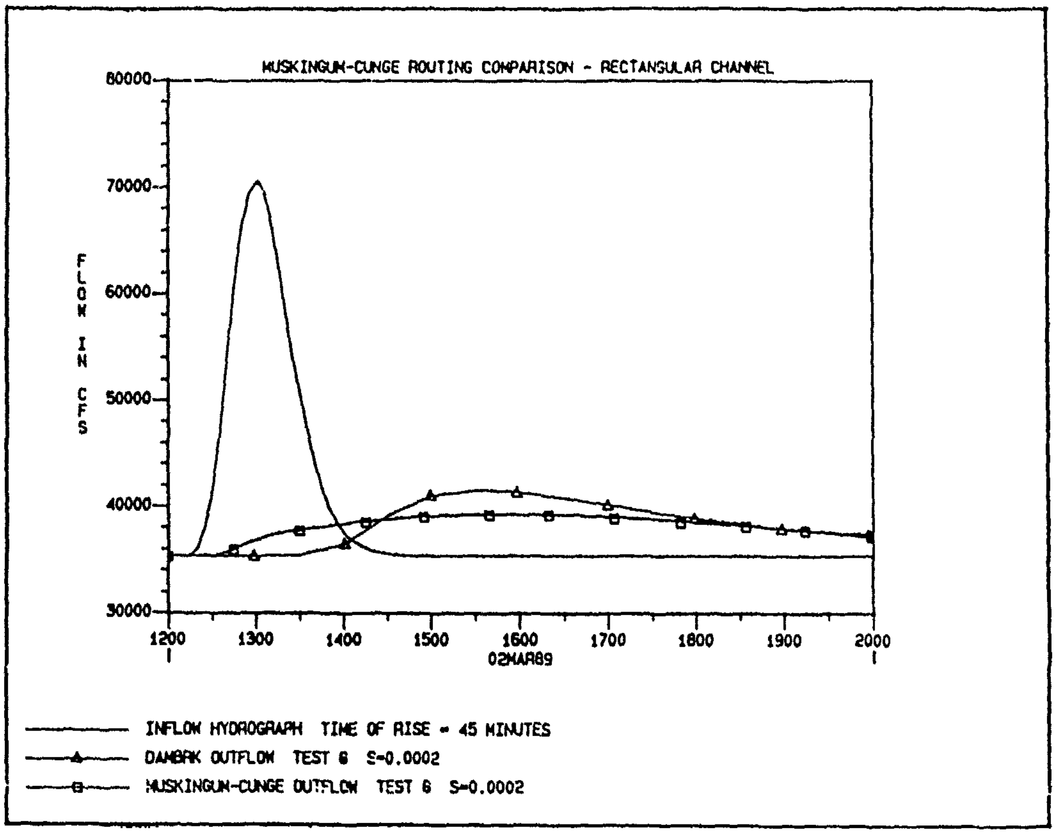


Figure 18. Rectangular channel, time of rise = 45 min, S = 0.0002

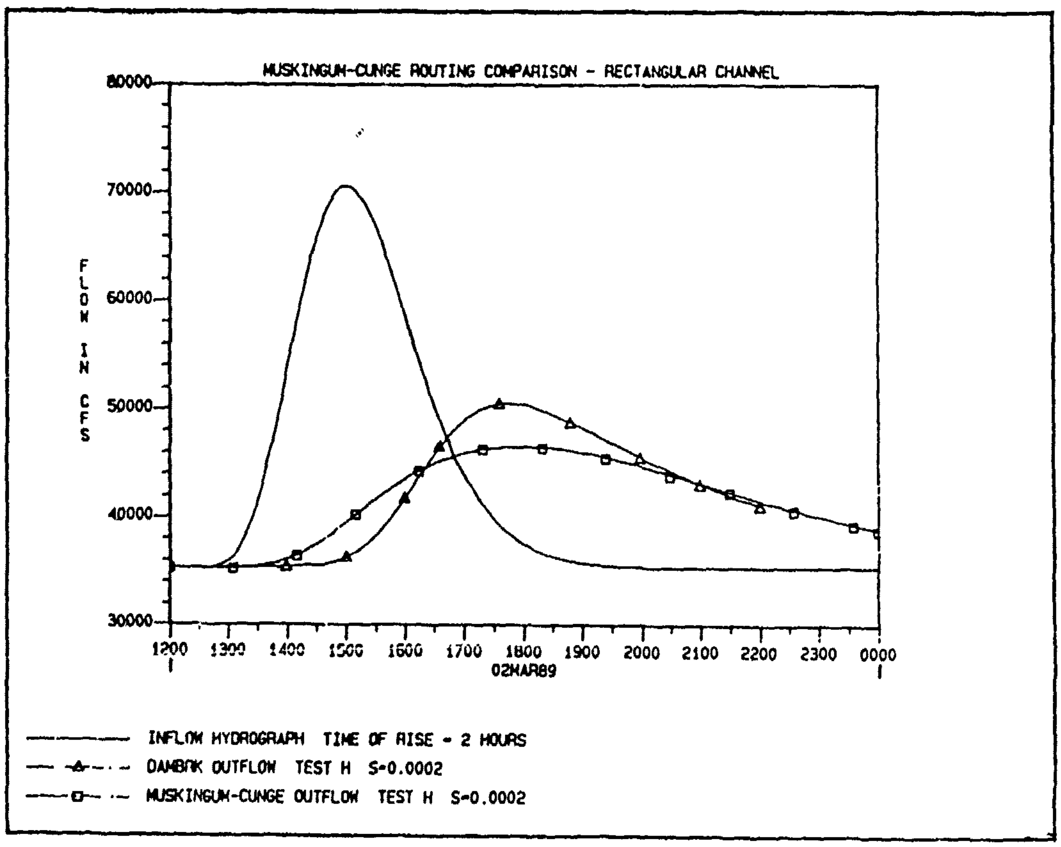


Figure 19. Rectangular channel, time of rise = 2 hrs, S = 0.0002

A third set of tests was performed for compound channel cross sections. A limited number of tests were run to analyze how well the Muskingum-Cunge method would compare to the full unsteady flow equations for channels with overbank flows. Shown in figures 20 through 25 are three different compound cross sections and the respective hydrographs from DAMBRK and the Muskingum-Cunge method. As shown in the plots, the Muskingum-Cunge method matches the DAMBRK hydrographs extremely well. However, for compound cross sections with very flat overbanks, the variable parameter Muskingum-Cunge method tends to lose volume. In general, variable coefficient methods have a tendency not to conserve mass. The error in mass conservation tends to be small (0 to 4 percent) and is not considered a significant problem.

The final set of tests compare the Muskingum-Cunge method with the traditional Muskingum method and the Normal Depth routing technique in HEC-1. The rectangular channel from the first series of tests (Figure 5) was used in this analysis. The resulting hydrographs are shown in figures 26 and 27. Both the Muskingum method and the Normal Depth routing technique had to be calibrated in order to match the results from DAMBRK. With the Muskingum method, it is necessary to calibrate all three parameters, K (travel time of the channel), X (weighting factor), and NSTPS (number of routing steps). The Muskingum method is considered a linear routing technique in that the parameters remain constant during the routing computations. Because of the linear nature of the traditional Muskingum method, it was not possible to match the shape of the DAMBRK hydrograph. This is evident in figure 25, where the traditional Muskingum method begins to rise much sooner than the DAMBRK and the Muskingum-Cunge hydrographs. This is typical of linear coefficient models.

The Normal Depth routing technique was able to match the DAMBRK hydrograph extremely well. The only draw back of this method is that the parameter NSTPS had to be calibrated. An equation for estimating NSTPS is provided in the HEC-1 manual. Unfortunately, this equation only ensures numerical stability during the computation, and does not guaranty accuracy.

VII. SUMMARY AND CONCLUSIONS

The numerical and physical basis for the Muskingum-Cunge channel routing technique were presented herein. This routing technique is considered a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are: (1) the parameters of the model are physically based, and therefore this method will make for a good ungaged routing technique; (2) the method has been shown to compare well against the complete unsteady flow equations for one dimensional flow; and (3) the solution is independent of the user specified computation interval. The major limitations of the Muskingum-Cunge technique are that: (1) the method can not account for backwater effects; and (2) the method begins to diverge from the complete unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections (i.e., channel slopes less than 1 ft/mi).

Channel length = 95,040 ft
Channel slope = 10 ft/mi

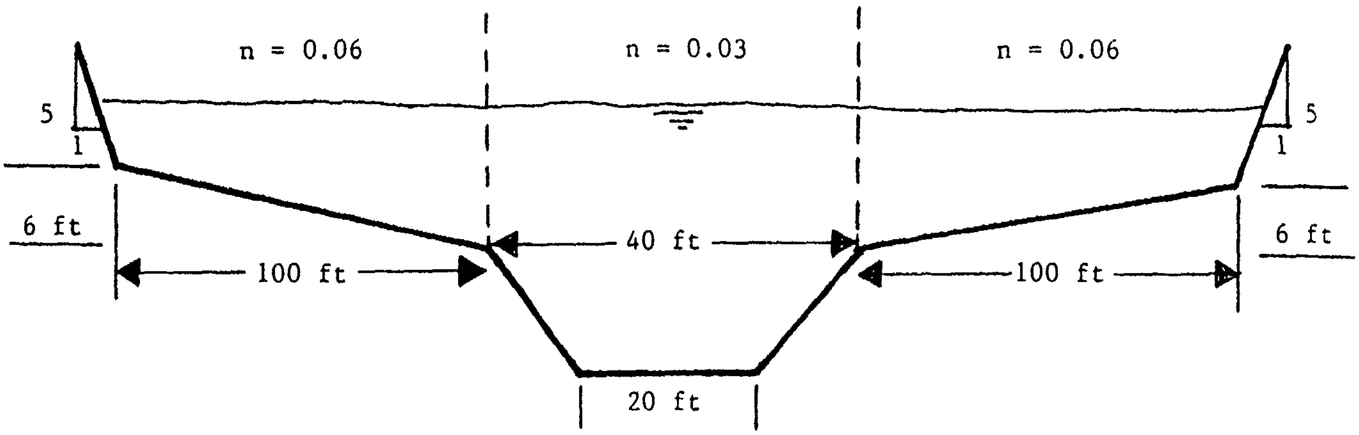


Figure 20. Compound cross section No. 1

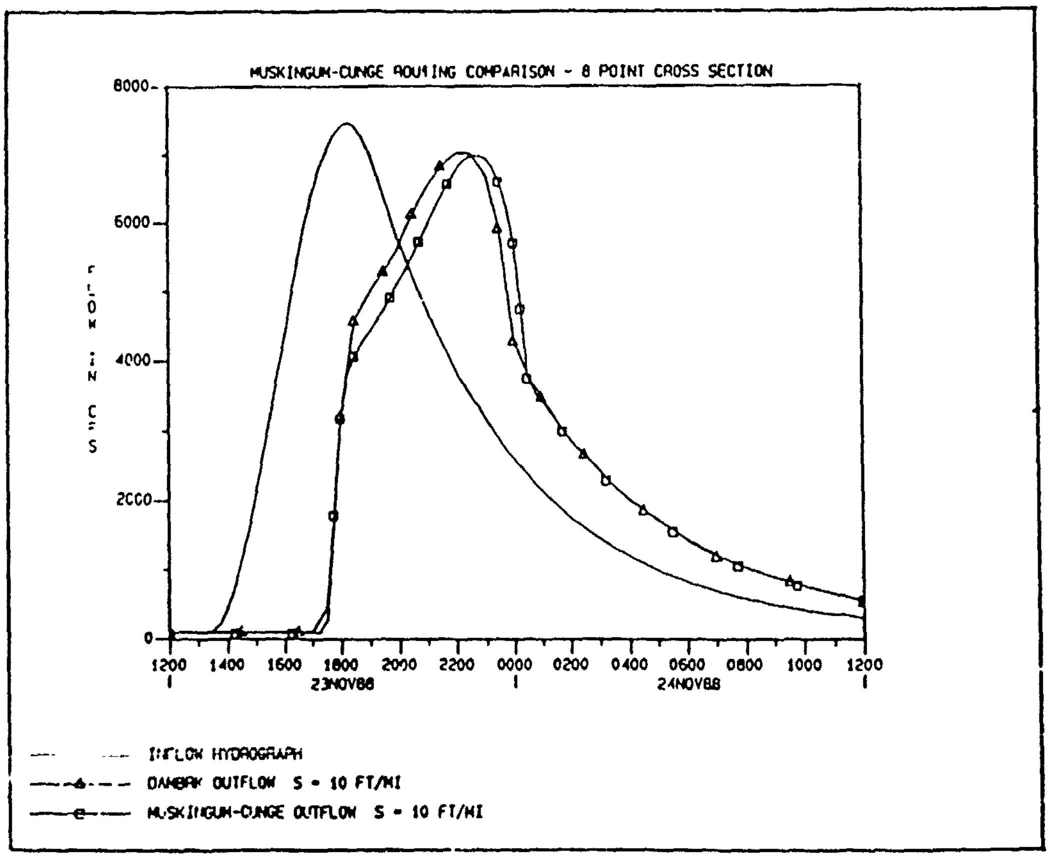


Figure 21. Resulting hydrographs from compound cross section No. 1

Channel length = 95,040 ft
Channel slope = 10 ft/mi

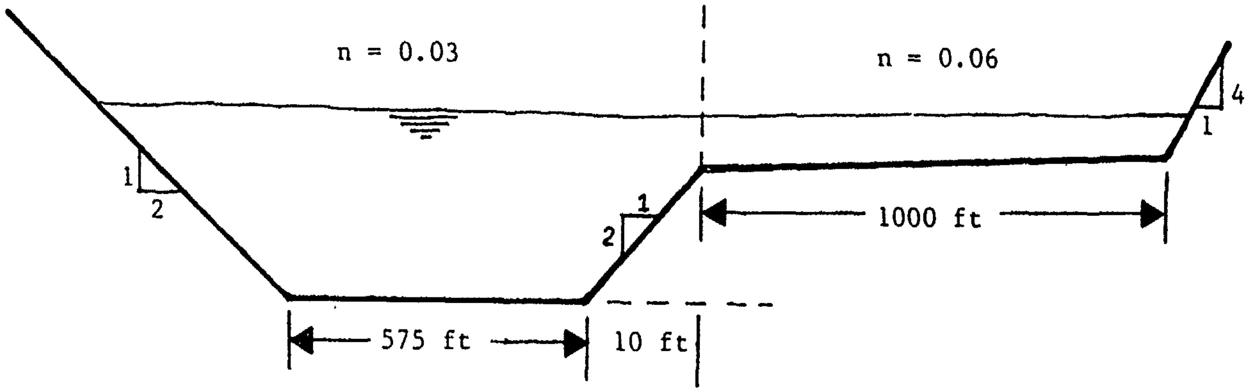


Figure 22. Compound cross section No. 2

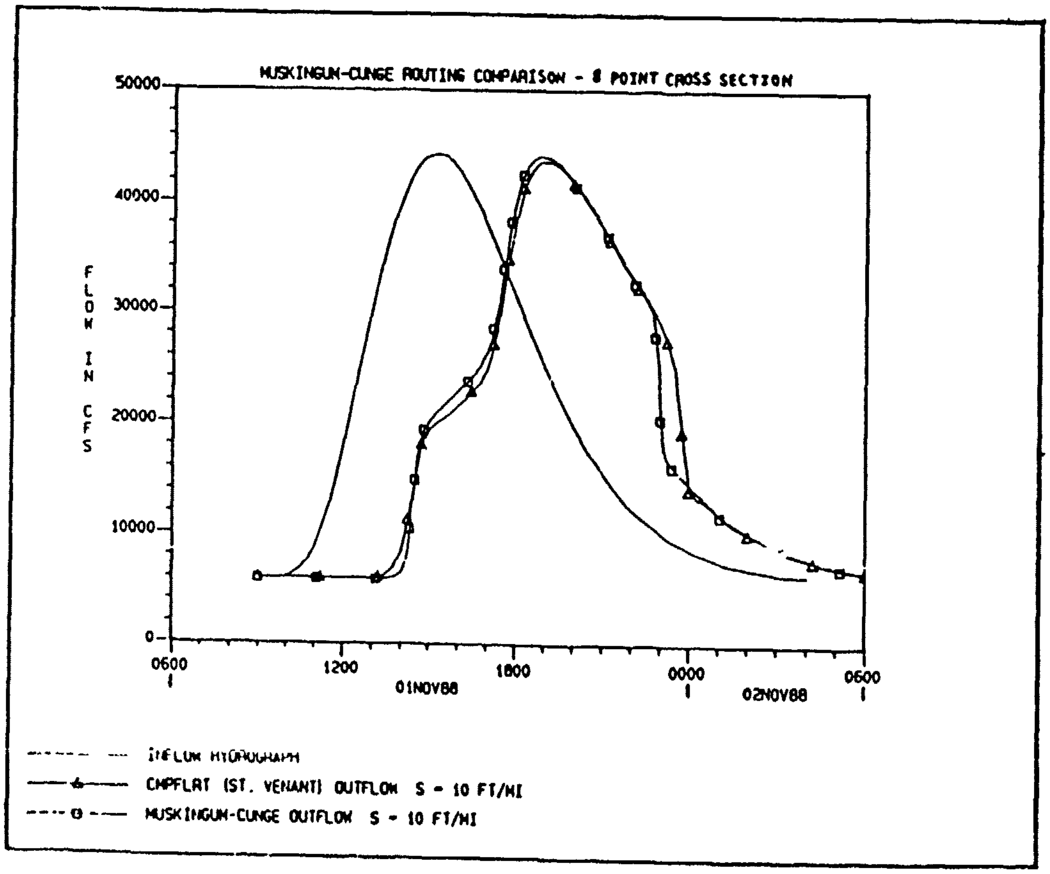


Figure 23. Resulting hydrpgraphs from compound cross section No. 2

Channel length = 82,025 ft
Channel slope = 10 ft/mi

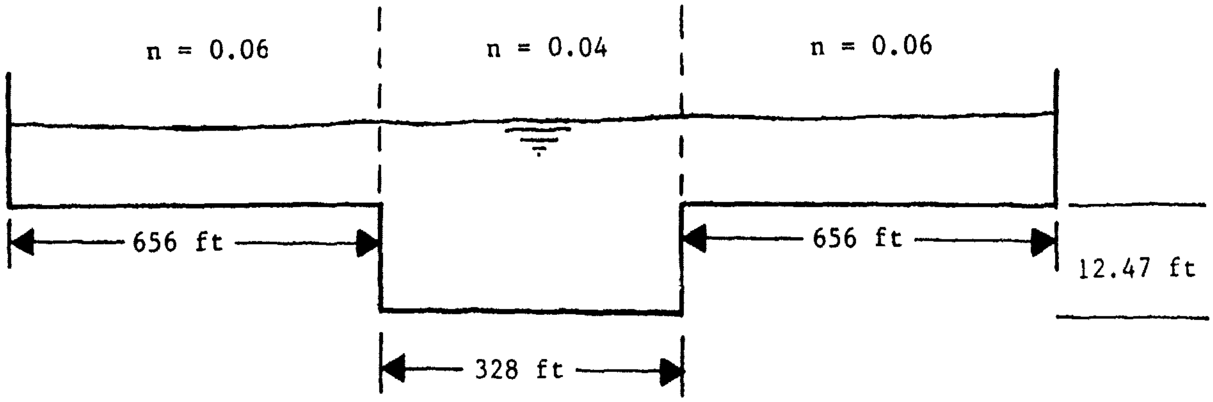


Figure 24. Compound cross section No. 3

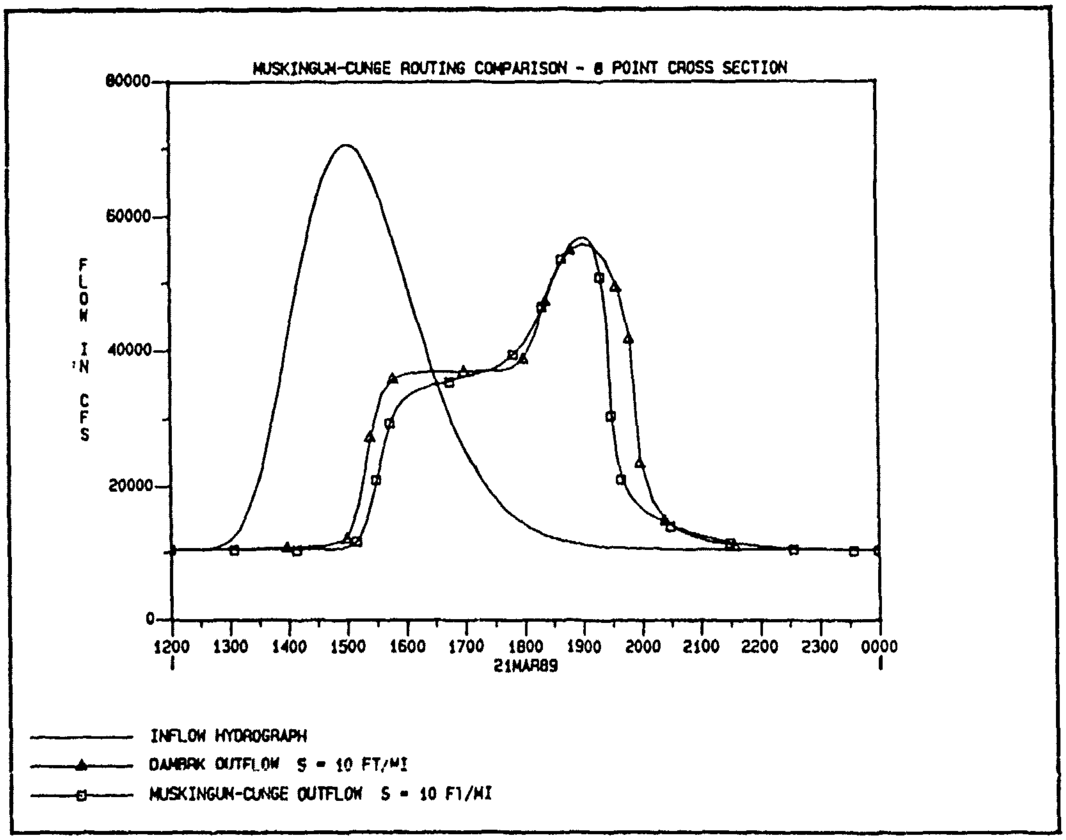


Figure 25. Resulting hydrographs from compound cross section No. 3

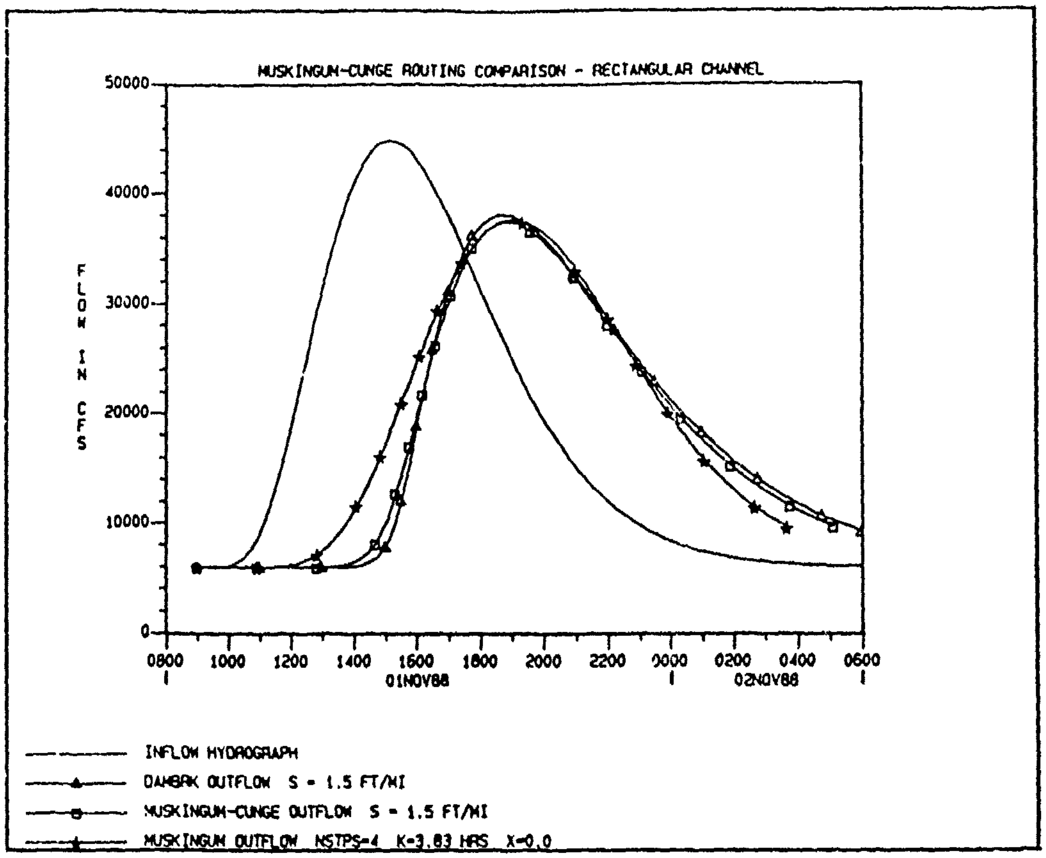


Figure 26. Comparison with traditional Muskingum method.

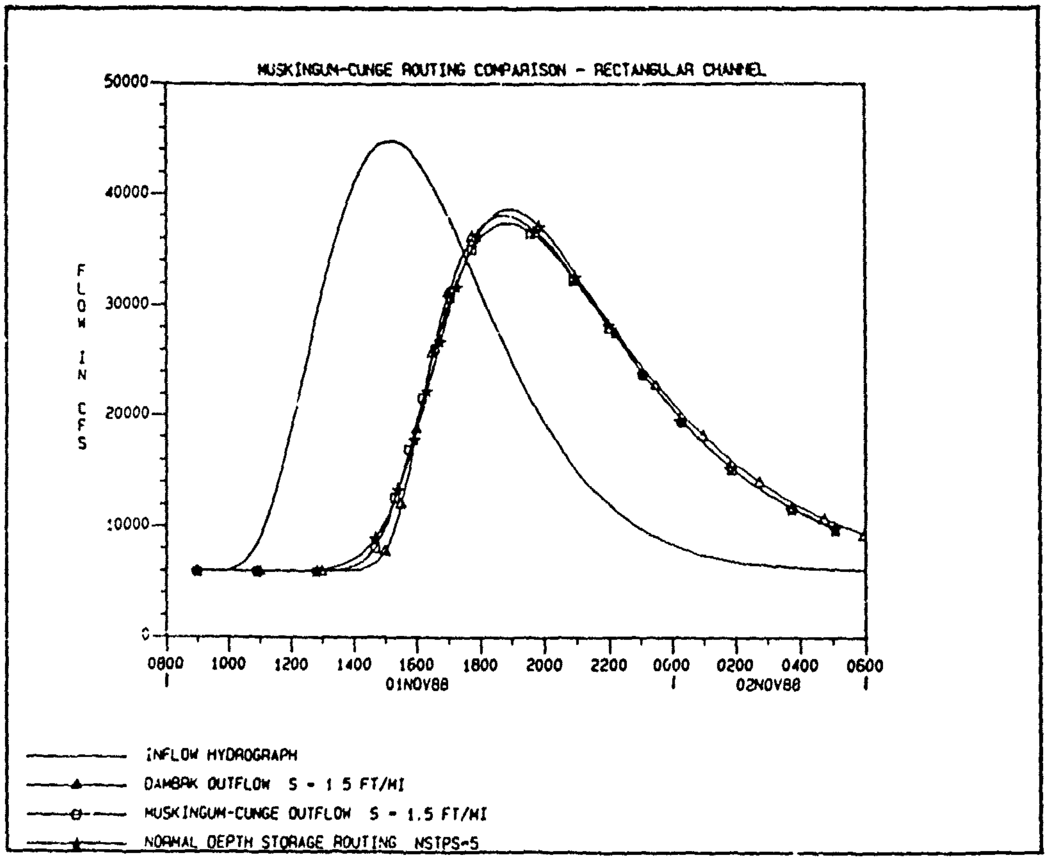


Figure 27. Comparison with Normal Depth storage routing.

VIII. REFERENCES

1. Cunge, J. A., "On The Subject of a Flood Propagation Computation Method (Muskingum Method)," Journal of Hydraulic Research, Vol. 7, No. 2, 1969, pp. 205-230.

2. Fread, D.L., "The NWS DAMBRK model," Theoretical Background/User Documentation, Office of Hydrology, The National Weather Service, Silver Springs, Maryland, June 1988.

3. "HEC-1, Flood Hydrograph Package, Users Manual, The Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, CA, September 1981, revised March 1987.

4. Miller, W. A. and J. A. Cunge, "Simplified Equations of Unsteady Flow," Unsteady Flow in Open Channels, Volume I, edited by K. Mahmood and V. Yevjevich, 1975, pp. 183-249.

5. Ponce, V. M., "Muskingum-Cunge Method With Variable Parameters," Journal of The Hydraulics Division, ASCE, Vol. 104, No. HY12, Proc. Paper 14199, December 1978, pp. 1663-1667.

6. Ponce, V. M., "Development of Physically Based Coefficients for the Diffusion Method of Flood Routing," Final report to the USDA, Soil Conservation Service, Lanham, Maryland, September 1983.

7. Ponce, V. M., "Diffusion Wave Modeling of Catchment Dynamics," Journal of Hydraulics Division, ASCE, Vol 112, No. 8, August 1986, pp. 716-727.

Muskingum-Cunge Channel Routing

by

Gary W. Brunner

SUMMARY OF DISCUSSION BY GARY R. DYHOUSE

Q: What is the definition of "rapidly rising"?
A: It's difficult, if not impossible, to give an exact answer. It really depends on the general slope and channel configuration of the reach under study. For the comparison tests shown in this paper, two inflow hydrographs were used, the first going from zero to about 70,000 cfs in first 2 hours, and the second peaking in 45 minutes with the same discharge. Both these hydrographs, when routed by Muskingum-Cunge, varied significantly from the DAMBRK results when introduced into a channel of one foot per mile slope. In this example, both hydrographs could be considered "rapidly rising".

Q: Is there any minimum incremental channel length for a routing reach?
A: We have tested routing reach lengths as short as 10 feet, with computational intervals measured in seconds. No problems with Muskingum-Cunge occurred.

Q: Have you tested the method in super-critical flow conditions?
A: No, the method is not applicable for these situations.

Comment: A very good analysis and comparison of the effects of a less-complicated technique to the full equations of motion. More of this sort of sensitivity testing and comparison should be performed by the labs.

CADD/GIS INTERGRAPH CAPABILITIES

by

Roger L. Gauthier[1]

1. Introduction

Implementation of CADD/GIS technologies within the Corps of Engineers, provides an opportunity for more efficient operations in hydraulics design, operational hydrology, and basin-wide planning. The Corps has embraced the utilization of computer-aided design and drafting (CADD) in an attempt to standardize operations in the construction and design arenas. Implementation of Geographic Information Systems (GIS) has been evolving within the Corps over the last 15 years in the attempt to enhance basin-wide planning studies, environmental impact assessments, and resource master planning. The CADD and GIS technologies are highly compatible, with the long-range goal being to create a corporate data base (CDB) structure, by mutual and timely implementation of both.

A CADD system uses map-level functions to portray features related to design and drafting operations. In the simplest sense, CADD can mean the automation of drafting operations in two- or three-dimensions. Automation of design operations are also afforded using civil and hydraulic engineering software packages. A computer mapping system (automated mapping/facilities management (AM/FM)) and certain land information systems (LIS) provide similar functionalities, such as display of the geographic locations of features for the drafting and production of maps only. CADD, AM/FM, or LIS in the simplest sense, do not provide innate information on the relationships with adjacent features (coincidence, proximity, etc.) nor any descriptive data about the features themselves. Products from these systems, hence, do not usually have native intelligence.

A geographic information system (GIS) is defined by the American Society of Photogrammetry and Remote Sensing (ASPRS) as the computer hardware and software used to input, store, retrieve, manipulate, analyze, and plot/print spatially (geographically) referenced digital data. GIS is an emerging technology within the Corps of Engineers and many Governmental Agencies nation-wide, causing a major review of institutional di[illegible]tions towards managing information resources.

1 Hydrologist, Detroit District, U.S. Army Corps of Engineers

GIS data frequently include information on land use, vegetation, soils, terrain, geology, hydrology, demographics, economic development, environmental parameters, and many others. The native intelligence provided in a GIS is based upon linkages between map features and tabular or descriptive data (attributes) usually stored in a relational data base management system (RDBMS). This linkage is called "topology," which has many forms and levels of sophistication.

In simplest terms, topology means that each map feature (line, point, polygon) has information directly attached to it which can be accessed on demand. For example, GIS "intelligent maps" can be queried by compound statements ("show me this, and this, but not this," etc.) to produce graphic representations of information required. Sophisticated topologies are being generated based upon "object-oriented" concepts, allowing the GIS to perform complex "what if" analyses. Topologies can be based upon spatial relationships or on temporal changes of a given parameter.

a. Purpose

This paper has been compiled to give a cursory overview of current Corps of Engineers initiatives to implement CADD/GIS technologies, through the consolidated procurement contract with the Intergraph Corporation, as they apply to hydraulics and hydrology applications. Due to the dynamic nature of hardware and software development, and new applications being developed over time, this cursory overview will rapidly become outdated.

The purpose of this paper is not to act as a corporate endorsement of the Intergraph Corporation, its products and software, but rather to simply discuss the potential impacts of the consolidated CADD procurement on H&H applications. Future computer system developments in the H&H arena may or may not be directly interfaced to the existing CADD contract or any follow-up to this procurement vehicle.

b. Key Issues

The key issue related to H&H applications using Intergraph hardware/software systems procured under the current CADD contract is integration. Integration of hydraulic design using CADD files generated by civil engineering, surveying, and geotechnical specialties is a clear example. Integration of hydrometeorologic observations in temporal and spatial GIS structures, used for improved water control operations is another example. Integration of basic physiographic, environmental, and socio-economic data in GIS's for basin-wide planning studies offers considerable benefits to the Corps' plan formulation and environmental analyses processes.

2. Definitions

The definitions given here are a cursory overview of key terms. A geographic information system can be any computer hardware/software system used to input, store, retrieve, manipulate, analyze and plot/print geographically referenced digital data. GIS's can be generated under either a raster (grid-cell) structure, or as vector maps, stored at user-defined scales.

A GIS can take many forms, from initial layered maps portraying specific themes, to sophisticated and highly intelligent data bases, using either relational or object-oriented topologies. Topology is simply defined as the computer linkages between map features and information about their spatial characteristics, temporal values, or relationships with adjacent map features. These types of information are frequently referred to as attributes of the given map feature.

Relational data bases are methods of structuring data so that relations between different entities or attributes can be used for data base queries and multi-parameter inventories.

Object-oriented topology is the most sophisticated form of GIS to-date. In these systems, an analyst can modify one map element or associated attribute, and get near-immediate changes in adjacent objects that are directly affected by the given change. An object-oriented topology appears to be the most useful tool for running computer models which modify one parameter over a range of occurrences (i.e., stage-inundation modeling based upon a range of water levels.)

3. Consolidated Procurement Highlights

The Corps of Engineers has developed a multi-million dollar requirements contract for standardized CADD computer systems, including hardware, software, maintenance, manuals, and training. This consolidated contract was awarded to the Intergraph Corporation of Huntsville, AL in 1987. It is intended to fulfill the design and drafting requirements of the Corps for Architect-Engineer and Construction (AEC) applications. Included within the broad category of civil engineering are structural, hydraulic, hydrologic, foundation, sanitary, road networking, and site planning, including the support functions of surveying and mapping.

The consolidated CADD contract, as awarded, covers hardware/software systems purchases through FY 92, with training and maintenance items continuing through FY 95. The contract also includes a technology upgrade clause, which allows for new hardware/software developments to be added over time, as these products are introduced on the marketplace.

a. Hardware

The consolidated CADD contract includes a wide suite of hardware platforms, including host minicomputers, high-end file servers, low- to high-end engineering workstations, networking hardware and software, and a variety of input/output peripherals.

The host minicomputers included on the contract are modified Digital Equipment Corporation MicroVAX computers. Most of the early CADD systems deployed within Corps offices opted for these systems. The price for the MicroVAX options ranged from $90K to $145K. With the advent of high-powered UNIX file servers on the CADD contract, many new users are opting for a pure networked environment. These servers include the Interserve 200, priced at $19K, which is a 5-MIP server, the Interserve 3005, priced at $38K, which is a 10-MIP server, and the Interserve 4200, priced at $62K, which is a 14-MIP server.

The lowest-end workstations on the contract are the Intergraph 225 workstations, priced between $14K and $35K, which are based upon a 5-MIP Clipper chip. The price differences reflect the chosen configuration and number of screens. These workstations are stand-alone UNIX computers, capable of operating in a networked environment or as local workstations to a host minicomputer.

The Intergraph 300-level workstations are mid-level, 5-MIP, single or dual screen (19"), stand-alone UNIX workstations, capable of operating in a networked environment or as local workstations to a host minicomputer. A single-screen workstation is capable of displaying 1 MB of data, while a dual screen can display 2 MB of data, each with 512 colors. The multitasking UNIX environment provides for operations using multiple windows, which allows the operator to run multiple CADD/GIS software operations concurrently. The Intergraph 300-level workstations price ranges from $28K to $52K, depending upon configuration and number of screens.

The Intergraph 3000- and 6000-series workstations, are 10- and 14-MIPS platforms, respectively. These systems can be purchased with either a 19" or 27" screen, with 1 MB and 2MB display capability, respectively, with a minimum of 1024 colors available. Image processing operations can be performed on either the 3000- or 6000-level workstations, even though higher level 6000 series workstations are preferred since they are capable of displaying image data in "true color" or with 16 million colors available. The Intergraph 3000- and 6000-level workstations range in price between $24K and $57K, based upon CPU, main memory, disk storage, screen(s) and digitizer configurations.

The CADD contract also provides for digitizing tablets which provide capabilities for inputting georeferenced data from large-format source documents such as topographic and planimetric maps and project drawings. The digitizing tablets can come with a "floating menu" and cursor. The CADD contract also provides for a suite of printer/plotters, to output data, in formats from 8x10" to E-size drawings (36x48"). Specialty laser printer/plotters are available which function in one of three modes; these being as a page printer, a screen copy device, and/or as a plotter.

The CADD contract also provides for a suite of input/output data devices (i.e., magnetic tape drives and CD-players). These device can be used for archival data storage, off-line file backups, software update transmission, and data exchange with other systems.

The Intergraph hardware is networked through IEEE 802.3 Ethernet communication hardware/software. These connections are essential for linking workstations for direct file exchange, resource sharing, and file management. The Intergraph hardware allows for interfaces to personal computers, utilizing Ethernet controllers, PC network file server software, and compatible graphic software drivers.

b. <u>Software</u>

The Intergraph software system developed to meet hydrologic engineering applications, being either CADD or GIS, is constructed in a modular basis. All software is based upon a core library of operating systems commands, running under UNIX System 5, which is shipped with each workstation. Intergraph's Microstation 32 software provides graphic display functionalities for both GIS and CADD operations.

The Intergraph GIS application software is available in two differing topologies. The Intergraph MicroStation GIS Environment (MGE) provides for "selective topology," being linked to a relational data base management system (RDBMS). Intergraph's Topologically Integrated GeoReferenced Information System (TIGRIS) software modules provide for object-oriented linkages between graphic elements and a RDBMS. The RDBMS can be either Oracle, Ingres, or Informix; all readily linked to the Intergraph GIS application software through a Relational Interface System, using concepts of the Structured Querey Language (SQL).

Additional modules are available in the MGE-family, including: Microstation GIS Analyst (MGA), which provides for selective query capabilities; Microstation Modeler (MsM), which provides for full 3-dimensional portrayals of terrain and hydrometeorologic data; Microstation GIS Translator (MGT), a package for importing basic physiographic and socio-economic data; Microstation Imager (MSI), which provides for full-featured image processing capabilities.

The Intergraph TIGRIS software, which has not yet been added to the Corps contract, has translators developed to upload MGE-based data files. TIGRIS should provide very powerful capabilities for true iterative modeling for the Corps.

The Corps' Geographic Resources Analysis Support System (GRASS), a fully functional GIS and image processing system, has also been ported to the Intergraph 300-series (and higher) workstations. GRASS has been development by the Construction Engineering Research Laboratory (CERL).

On the CADD-side, the consolidated contract includes Intergraph's InRoads application software, which is a combination of the InSite and InFlow application software packages. InSite provides for advanced applications including civil siting, coordinate geometry, terrain modeling, surface display, cross section extraction, profiling, and earthwork quantity computations. Inflow provides for drainage system layout and network design, open channel modeling, hydrograph generation, and profile capabilities.

The consolidated contract also offers network file management and file security software which runs in a network environment.

4. Data Base Considerations

A critical concern within the GIS community in the past has been differences between raster and vector data bases and transportability of information between them. Generally, raster structures provide better analytic capability while vector structures provide better map resolution and generation. It is now acknowledged that both data structures are valid methods for representing spatial data, and considerable software development has occurred to use both structures on the same hardware platform.

In order to support remote sensing data analysis and to incorporate CADD data, the GIS structure used for H&H operations must support some form of both raster and vector processing and display. For example, most GIS's use USGS digital base map information, distributed in Digital Line Graphs (DLG's), which are vector files, and USGS terrain data distributed as digital elevation models (DEM's), which are raster files. The Intergraph hardware/software outlined in this paper fully supports both raster and vector functionalities.

A large problem still remains on transporting attribute information which is tied to map features (e.g., from Info to INGRES to ORACLE to INFORMIX, etc.) Relational interfaces and structured query language (SQL) development, however, are reducing this problem as time goes by. Work is continuing on data base formats and data exchange standards, but serious problems still exist in these areas with the various formats used.

The Intergraph CADD/GIS hardware/software outlined herein has a very robust data translation capability. Digital spatial data over large regions of the U.S. are being generated by numerous other Federal agencies and by others using standard vendor-provided formats. Much of this information is capable of being directly transported onto an Intergraph system. CADD design files and GIS files are highly compatible, facilitating information exchange between these differing user groups.

Since the principal reason for CADD or GIS applications for H&H operations is to be able to make better management decisions, it is essential that the initial data must be sufficiently reliable and error-free for the purposes for which they are required. As data are developed for a specific project, the scope and resolution of the data needed are usually determined by the specific project applications. Although challenging, additional efforts need to be applied for considering future needs and possible uses.

Whether data are developed in-house or by contract, the data must be checked for logical consistency, accuracy of position, and accuracy of categorization. For those data created by contractors, random points on the produced data should be checked selectively against the original data source.

An essential part of each data set is the "audit trail" of the steps involved in measurement, interpretation, analysis, etc. of the encoded data. This "data genealogy" must be imbedded in the data base and should include such items as the source manuscript, procedure(s) used, accuracy, error standards, source materials, name of the technician or analysts, and date on which the data was entered. Any subsequent modifications to or manipulations of the data must be similarly recorded. It is essential that data dependably reflect stated accuracy standards and that analysts and modelers understand the implications of the accuracy inherent in the data.

5. Potential Hydraulics and Hydrology Applications

A. Water Control Data Systems

A basic mission assigned to most Corps Districts is the collection, analyses, coordination, and dissemination of basic hydrometeorologic data, including water levels, pertaining to operations of water control operations. Most data collection operations, and many output products, which routinely required considerable effort to produce manually in the past, are now fully automated under existing water control data systems (WCDS) computer networks.

Improvements and enhancements are needed in the next-generation of the WCDS in graphic output and development of interfaces between basic hydrometeorologic data collection and water supply, inflow/outflow, and regulation models. These upgrades could be realized by utilizing GIS technologies. The next generation of the WCDS is presently under design by the Corps, with consideration being given to utilizing the existing CADD contract as a prospective procurement vehicle.

B. Basin-Wide Planning Studies

Flood area determinations, frequently required for basin-wide planning studies, have been generated from computer analyses of satellite imagery by various offices including the Detroit, Fort Worth, Pittsburgh, and Vicksburg Districts, and by the Environmental Laboratory of WES, and stored in GIS thematic layers. These data are also useful for economic evaluations, FEMA flood insurance studies, and emergency operations.

A Great Lakes Shoreline GIS is being developed by the Detroit District to support environmental, socio-economic and hydrodynamic assessments of the IJC Great Lakes Water Level Reference Study. The framework for a comprehensive GIS system for the Great Lakes is being designed to contain physiographic, demographic, and economic information of the U.S. Great Lakes shoreline. Emphasis is being placed on incorporating environmental and economic data for modeling erosion/recession, wetlands changes, and storm-surge flooding caused by fluctuating Great Lakes water levels.

C. Other H&H Potential Applications

Use of GIS in runoff modeling is being developed by the Hydrologic Engineering Center (HEC), with translators being written to read Intergraph map data directly as model inputs. Use of GIS to automatically generate / update cross section data for generation of flow profiles is also being examined. In short, GIS and hydraulic / hydrologic analyses are beginning to overlap and complement each other.

H&H support of emergency operations and preparedness, including interior stream and coastal flooding risk assessments, required protection, and response plans can also benefit from application of CADD/GIS capabilities.

The Corps is involved in Hazardous and Toxic Waste (HTW) mitigation and cleanup nation-wide. Application of GIS technologies to this mission will be significant, particularly for tracking data at various sites and for modeling cleanup strategies. Toxic sediment and chemical data, well drilling information, boring log data, survey data, and historic information (structure locations, disposal sites, etc.), derived from aerial photography and maps, all would be important inputs for a GIS.

Underlying water table data (such as geologic stratigraphy, depth and thickness of aquifers, areal distribution of tranmissivities, location of faults, etc.) all can be stored in GIS data layers in much the same fashion as the other layers of

thematic data of surficial features. Overland and subsurface (aquifer) multivariate analyses and flow modeling would be a natural result of the compilation of this data in the GIS.

CAD/GIS Intergraph Capabilities

by

Roger Gauthier

SUMMARY OF DISCUSSION BY GARY R. DYHOUSE

Q: How are you going to fund the maintenance of the data base?
A: This is possibly the biggest problem with the system. No continuing source of operation and maintenance funds are available for this purpose. While the initial development of the data base usually is by project funding, no provisions are included for the continuous updating and maintenance required. Of equal concern is how to insure the integrity and quality of the data which is periodically included in the base update.

Q: How difficult is it to get people trained in the use of CAD?
A: The vendor offers numerous training courses and we have had individuals attend up to seven different classes. However, the software has become much more user-friendly, and a considerable amount of self-instruction is now possible. Another problem we have experienced is the ability to retain skilled CAD personnel. OPM standards do not address these skill levels or even an occupation series. These people are in great demand outside of the Corps.

WEATHER RADAR

by

Thomas L. Engdahl[1]

INTRODUCTION

A number of important hydrologic process components can be improved in the model simulation process. These include rainfall, infiltration, runoff generation and channel flow (Feldman, 1987). Of these components, real-time hydrologic forecasts models appear most sensitive to the spatial and temporal distribution and amount of rainfall (Barrett, 1985). Thus, an improvement in rainfall measurement and its spatial distribution is an important factor that could lead to improved flood forecasts in real time. Ideally, a capability for accurately forecasting precipitation would make an even more significant contribution, but such technology has not been adequately developed for operational use in hydrological forecasting.

DATA AVAILABILITY

The use of weather radar information is being demonstrated in the Inland Waterways Remote Sensing Demonstration Program being conducted at the Rock Island District. National Weather Service WSR-57S and 74S 10.3 cm and WSR-74C 5.4 cm weather radars are being used to monitor storm events. Storm events are being collected at the Rock Island District using an Alden C2000R weather radar receiving unit with a serial port. The individual weather radar scenes are being processed using a MSDOS 386 microcomputer.

To process weather radar data effectively, a digital form of the scene must be used. A digital video integrator processor (DVIP) is used to analyze the reflected radar signal and determine its intensity. The DVIP converts radar reflectivity returns into digital form, averages several returns and then integrates many of these returns over time. These range-normalized reflectivity values are gated to obtain threshold values for rainfall rates. The output from the DVIP is a raster array with a video integrator processor (VIP) level associated with each pixel within the array (Miers and Heubner, 1985). Each VIP level corresponds to a range of rainfall rates;

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[1]Civil Engineer, Environmental Laboratory, USAE Waterways Experiment Station

values for convective storms are tabulated below, along with values used in the radar-rainfall conversion models.

	Rainfall Rates (cm/hr.)	
VIP Level	Nation Weather Service (NWS) Range	Model Value
1	0.0-0.5	0.1
2	0.5-2.8	1.4
3	2.8-5.6	4.1
4	5.6-11.4	8.3
5	11.4-18.0	14.5
6	>18.0	21.4

APPROACH

A schematic showing the stand alone methods used to receive, display, analyze, and calibrate weather radar scenes is shown in Figure 1. The VIP level raster arrays are received at a weather

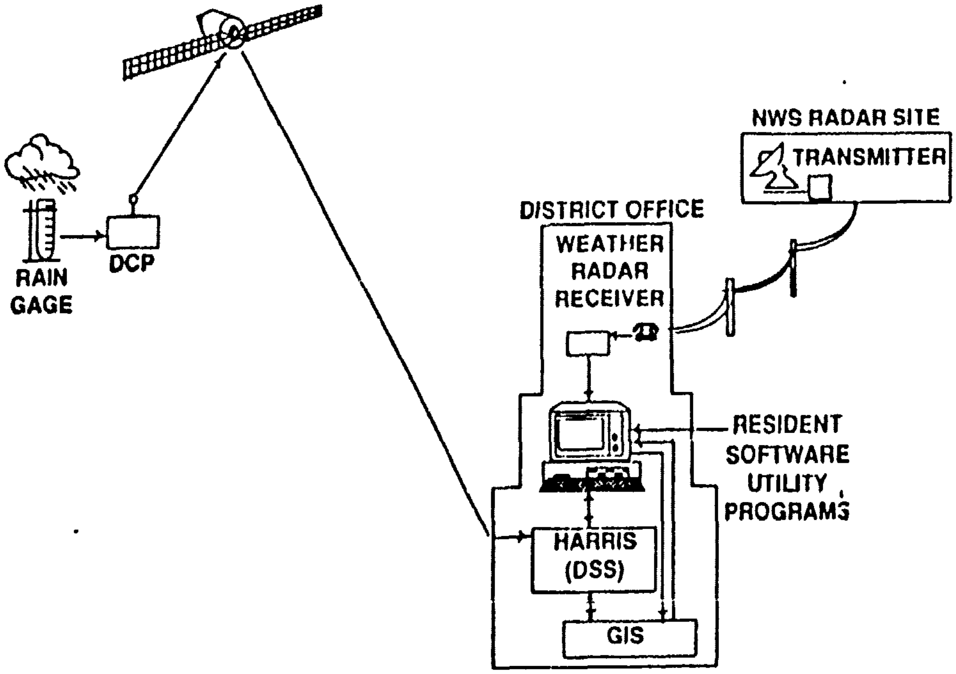


Figure 1. Automated Weather Radar Data Acquisition System.

radar receiving unit via telephone lines. As soon as the arrays are received and displayed at the receiving unit, they are sent to a microcomputer for filing, processing and display. Each scene is given a unique file name, VIP levels are converted to rainfall amounts, and hourly cumulations of rainfall are displayed. River basins within the scan radius of the weather radar which have been digitized and rasterized are also displayed on the microcomputer. Each hour, displays showing integrated rainfall amounts over preselected subbasins are generated and Data Storage System (DSS) files are made for each subbasin.

During or after the storm event, programs can be run which access the DSS file containing the hourly rainfall amounts recorded at the gauges in the field. Those gauges within the scan radius of the weather radar are used to calibrate the weather radar scene each hour. The hourly calibrated weather radar scenes are integrated over the subbasins of interest, and DSS files are generated and displayed on the microcomputer.

STUDY RESULTS

The capability to monitor and convert weather radar scenes to rainfall amounts in real time has been demonstrated. Calibration programs have been developed to utilize rain gauge information during or after the storm event. The primary problem in utilizing rain gauge information in real time is the ability to communicate to the resident DSS file which for most districts is located on the Harris 1C) mainframe computer. During the demonstration program, the rris was not networked to the microcomputer, and an opera. r had to physically run communication programs to extract rain gauge information from the Harris to the microcomputer. In addition, the rain gauge information lagged behind the weather radar data by four hours due to the burst frequency of the Data Collection Platforms (DCP).

Comparison of mass curves and hyetographs for individual subbasins for both calibrated and uncalibrated weather radar data have been made for approximately five storm events within the Rock Island District. These curves and hyetographs indicated that weather radar calibrated data may improve short term hydrologic forecasts. The next step in this analysis is to run weather radar estimated rainfall data through hydrologic models to show the impact of this data on river volume, timing and duration as opposed to using rain gauge only information in Corps hydrologic models.

CONCLUSIONS

Weather radar data can be used to estimate the spatial and temporal patterns of rainfall events in real time. Stand alone software routines have been written which allow Corps district offices to monitor and analyze weather radar data, and with operator assistance, allows offices to calibrate weather radar data to telemetered DCP rain gauge data.

The weather radar data must be run through hydrologic models to understand the impact of spatially distributed radar/rainfall data on hydrologic forecasting. An analysis to determine this will be conducted this summer by the University of Iowa using three different hydrology models.

REFERENCES

Barrett, E.C., "Rainfall Evaluation by Remote Sensing: Problems and Prospects", Hydrologic Applications of Remote Sensing and Remote Data Transmission, Publication No. 145, 217-258, 1985.

Feldman A., "HEC Models for Water Resources Simulations; Theory and Experience", Advances in Hydroscience, Vol. 12, 35-50, 1987.

Miers, B.T., and Huebner, G.L., "Military Hydrology; Report 8, Feasibility of Utilizing Satellite and Radar Data in Hydrologic Forecasts", Miscellaneous Paper EL-79-6, prepared by U.S. Army Atmospheric Sciences Laboratory, White Sands Missile Range, New Mexico, for the U.S. Army Engineer Waterways Experiment Station, 51, 1985.

RADAR APPLICATIONS

by

Carroll E. Scoggins[1]

Introduction

We, in the Southwestern Division, have been interested in using radar information for real-time reservoir control for several years. Because we are located in a part of the country which has frequent and severe thunderstorms, several federal and state agencies in this area have been very active in trying to improve the radar information for a number of different uses. Because of the amount of activity in this area and our involvement with the various agencies, the Southwestern Division (SWD) and the Tulsa District have been assigned as the lead division and district in development of the Corps use of the Next Generation Radar System (NEXRAD). The Hydrologic Engineering Center (HEC) and the Waterways Experiment Station have also been very active in developing methods and techniques for using radar information. This paper will focus on Corps access to the system and radar applications to reservoir control and hydrologic engineering. Charlie Sullivan, SWD, is the Corps point of contract for the NEXRAD project. Working for Charlie is Steve Fortenberry, a meteorologist with the National Weather Service (NWS), presently assigned to SWD. All coordination with other agencies involved in the NEXRAD project should go through SWD at this point in time. Within the Tulsa District, Clinton Word is the point of contact. Steve has made several technical presentations on NEXRAD and most districts or divisions already have someone familiar with the NEXRAD project. The NEXRAD project may very well prove to be one of the most significant improvements in data acquisition for real-time reservoir control in recent history.

WSR-57 (Weather Service Radar 1957) Radio Detecting and Ranging (RADAR) has been used to track rainfall producing storms for some 33 years. During this time the Tulsa District has used radar data as an aid but never in actually making forecasts of inflows to reservoirs. The Tulsa District first used radar data received over the NWS RAWARC line in the late 1950's. This data was handplotted on a map and generally consisted of the location, size, speed, direction of the storms, percentage of the area covered by storms, and the elevation and location of the primary cells. An example of these plots is shown in enclosure 1. In the late 1970's, radar information could be received over a telephone from a specific site. All that could be shown was the location, the relative intensity, and geographic limits of echo producing clouds. Also in the late 1970's, information from the NWS D/RADEX project was accessed where available and consisted of a series of numbers and letters on a circular grid depicting estimates of rainfall over a period of time as shown in enclosure 2. This information was calibrated to a certain extent and used primarily to determine where intense rain was falling but was not considered reliable enough to actually use for forecasting. An expanded version of the D/RADEX project called the RADAP (Radar Data Processor) and later the RADAP II system continued to develop in the 1980's. The Oklahoma City NWS office made significant improvements in the quality of the data by placing equal priority on monitoring rainfall and tornadoes. Formerly, when tornadoes were present, they were followed exclusively, resulting in gaps of several hours between radar sweeps through the clouds and even more unreliability of the radar rainfall estimates. Even with the new priority, the quality of the data was still considered too unreliable to use in real-time forecasting because of the limitations and uncertainties associated with the WSR-57 radar and calibration.

[1]Chief, Hydrology and Hydraulics Branch, U.S. Army Corps of Engineers, Tulsa District

Current System

The Tulsa District accesses the Oklahoma City area RADAP system through our Water Control Data System (WCDS) every 3 hours. The data is used as a guide as to the quantity, timing, and aerial extent of rain. Other RADAP sites in this area are located at Wichita and Garden City, Kansas; Monett, Missouri; and Amarillo, Texas; but are not accessed on a regular basis. Because of the development of the WSR-88D (Weather Service Radar 1988 Doppler) or NEXRAD system, very little effort has been made to improve the quality of the data from any of these RADAP sites for the last 2 years. In addition, we access radar from a commercial vendor through our WCDS system daily and can get updates no more than 90 seconds old from individual radar sites or 30 minutes old for composite pictures. These are displayed on overlays of satellite imagery or on map backgrounds. Several other weather products are obtained and distributed to other elements in the District through our office automation system. Several districts in the Corps obtain radar information through a commercial vendor similar to this.

NEXRAD

Development of NEXRAD is the result of a tri-agency program involving the Department of Commerce (National Weather Service), the Department of Transportation (Federal Aviation Administration), and the Department of Defense (Air Weather Service and the Naval Oceanography Command). The administration of the NEXRAD program is assigned to the Joint System Program Office. They have the responsibility for acquiring the NEXRAD system and making it operational. Although the Corps was not a part of the initial development of NEXRAD, the Corps now will have access to NEXRAD sites thanks to the Air Weather Service arm of the Department of Defense. Development of the NEXRAD system is being done in four phases. These are: 1) the system definition phase, 2) the validation phase, 3) the limited production phase, and 4) the full scale production phase. An operational support facility was constructed at Norman, Oklahoma, and testing under phase 2 is nearing completion. The limited production phase consisting of 10 NEXRAD units will begin this year (enclosure 3) followed by the full production phase which will install four units per month. Some 175 sites have been identified as shown on the map (enclosure 4) and completion is scheduled in 1995.

Enclosure 5 shows the main components of the WSR-88D system. High technology S-band doppler weather radar units will be connected directly to a powerful digital computer called a Radar Product Generator (RPG). These RPG's will process the raw reflectivity and doppler information to provide visual and digital products. The visual products are designed to be displayed on Principal User Processors (PUP's) which are sophisticated graphics display computers. The operators of the PUP's may view, manipulate, and re-process the visual products in a variety of ways.

Communications

Corps access to the system will be via a dedicated port in each RPG. Configuration of the hardware and software is well underway. A NEXRAD communications working group is planning on networking all the RPG's. Since some aspects of the NEXRAD communications is still not defined and will not be available for a year or two, the Corps has opted for development of a PUP emulator for the immediate future. The contract specifications for this Principal User Processor Interactive Emulator (PUPIE) was developed by HEC. The PUPIE will perform the necessary operations in hardware/software to remotely access multiple NEXRAD sites, communicate with the RPG system at the site, request NEXRAD products, retrieve and store products, and display certain graphical products. The initial implementation is targeted to operate in industry standard Intel 386 AT-bus hardware, using the SCO UNIX 386 System V operating

system. The PUPIE is being configured as three functional modules. The first and third phases are communications modules (COMRAD and COMRAD5) and will perform all functions necessary to communicate with RPG's. The second phase is the view model (VUERAD) and will perform all the functions necessary to view certain NEXRAD graphics products. The contract for the first phase, COMRAD (Communications Module) will be let within a month and is scheduled to be completed by October 1, 1990. This will allow the Corps to obtain the hydrologic data in digital form. Award of phases II and III will be dependent on the availability of funds. Plans also are for the Corps to develop the capability to access the RFC PUP's as a backup. Similarly the RFC's may wish to backup their system by accessing NEXRAD data from the Corps.

Radar Applications

Several questions arise when considering the use of radar information in hydrologic engineering. These include: 1) What data is available, 2) How good is it, 3) What are the applications, and 4) What will it cost?

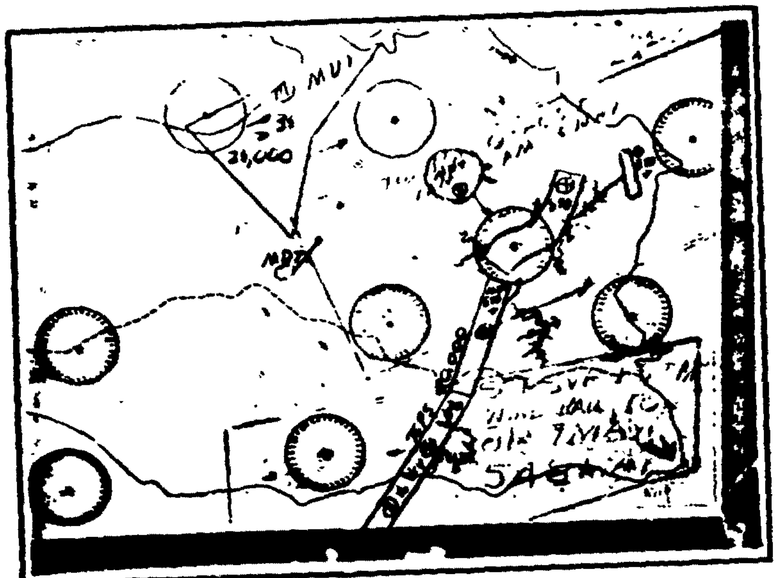
1. What data is available? A flow chart showing the base data, the algorithms, and the products are shown in enclosure 6. The Corps principal interest will be in the algorithms and products generated from the reflectivity base data. Precipitation information, both accumulated and projected, will be available for various durations and aerial extent throughout the day. Storm characteristics such as tracking, structure, and echo tops will also be available along with severe weather probability.

2. How good is it? The NWS will calibrate the precipitation estimates with information from rainfall gages. Plans call for utilizing up to 30 gages for this calibration. The accuracy of this calibration will depend to a large extent on the quality of the ground truth data used for the calibrations and will vary from place to place across the country. Dr. Ken Crawford, State Climatologist for Oklahoma, has proposed a mesonetwork (automated environmental monitoring systems across Oklahoma) which, if implemented, would: a) install and activate a dense network of 107 automated weather stations across Oklahoma, b) observe agricultural, hydrological and meteorological conditions every 15 minutes, c) process the data and make the resulting value-added information available to a variety of statewide users within minutes of each observation time, and d) process and assemble the data to provide climatological information over time period ranging upwards from a day to a year or longer. The Oklahoma Law Enforcement Telecommunications System will be used as the backbone of this proposed network. A schematic of the proposed mesonetwork is shown in enclosure 7. Obviously, a system like this would be of substantial value in improving the accuracy of the radar calibration but what about areas where no such system is available? In many locations the rainfall data obtained from Corps data collection platforms may be the best information available, at least initially. If so, stations should be graded for accuracy and those stations selected for calibration should have their reporting frequency changed according to the need. As NEXRAD becomes available in different parts of the country, the Corps should work with the NWS and the various federal, state, and local agencies to develop as reliable a calibration system as possible. There still will exist uncertainties in the radar data due to limitations in the radar capability such as errors due to curvature of the earth, anomalous propagation, physical barriers, and several other factors present in radar technology. However, with the advent of the doppler technology and the density of the coverage in many locations the accuracy in the radar data alone will be greatly improved over the existing system. Only by comparison with known data at various points will a user be able to determine if the data is accurate enough for his intended use.

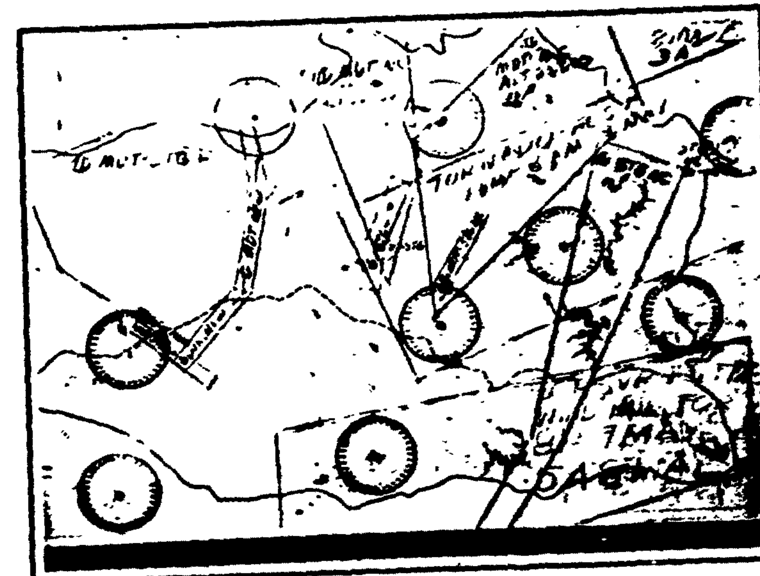
3. What are the applications? Once the accuracy question is resolved, the data should be extremely valuable for forecasting inflows into reservoirs and for making reservoir control decisions during flood periods. For the first time, precipitation data will be available throughout

the day on an hourly basis. In addition, the geographical limits of the rain and distribution will be defined. The data will be available on a grid basis which can be processed and entered into a forecasting program such as HEC-1F. Work is presently being undertaken by HEC to: a) pre-process the information so it may be input into HEC-1F, b) modify HEC-1F to accept the NEXRAD data directly, or c) develop hydrology models that take advantage of the large amount of precipitation data that will be provided. In addition to helping make real-time reservoir control decisions, the data can be archived for hydrological engineering studies of historical events. This data should be of tremendous value in reconstructing the rainfall for hydrologic model development and calibration, and for defining rainfall and loss rates.

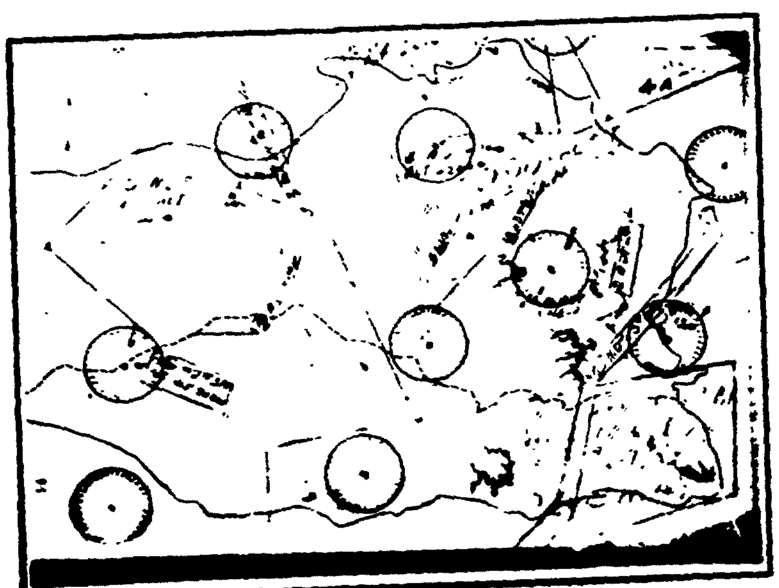
4. What will it cost? At this point in time the cost of accessing the RPG looks very nominal. In order to access a site, a modem compatible with the RPG must be installed. A 386-PC will be needed at the receiving site to be compatible with the PUPIE. Presently, we do not anticipate any user charges for the Corps of Engineers. For information concerning equipment and procedures, contact Steve Fortenberry at 214-767-2393.



(a) Radar Scan 2 a.m.



(b) Radar Scan 3a.m.



(c) Radar Scan 4 a.m.

(d) Rainfall Reported at 7a.m.

PHOTO RECORDS OF RADAR SCAN DATA AND REPORTED RAINFALL EARLY MORNING 8 MAY 1961

ENCL 1

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$K
A--3 HR RUNNING TOTAL
B--PREVIOUS 3 HR
C--CURRENT 3 HR
D--PREVIOUS 24 HR
E--CURRENT 24 HR
F--CURRENT STORM TOTAL
>>B
CATEGORY SIZE(100THS OF AN INCH): 2
THRESHOLD: 1
TYPE "A" FOR CROPPED GRID
EAST-WEST MILES: A

CRW  AVERAGE ACCUMULATED RAINFALL MAP


CODE VALUE        1   2   3   4   5   6   7   8   9   A   B   C   D   E   F
RAIN(100TH IN)    2   4   6   8  10  12  14  16  18  20  22  24  26  28  30

    120 108  96  84  72  60  48  36  24  12   0  12  24  36  48  60  72  84  96
      W...W...W...W...W...W...W...W...W...W...*...E...E...E...E...E...E...E.. E.  E

115N                         1                                 *
110N                      *  1                                    *
105N                   *                                            1*
100N                 *                 * * * * * * * *               1 *
 95N               *             * *                     * *             *
 90N             *            *                              *             *
 85N           *           *                                    *            *
 80N          *         1*1 11 1                                  *   21 2 1  *
 75N        *          *                * * * * * * *   --11 1 222  *1322432112
 70N       *         *             * *                 * *1 21        *  11
 65N      *         *           *                          *            *
 60N     *         *          *                              *           *
 55N    *         *         *                                  *          *
 50N             *  1111232111212211     * * * * * *             *         *
 45N   *        * 2 113343112212111 1*                *           *         *
 40N  *        *   12779414322211112121 1               *           *        *1
 35N  *       *     13A82CD85554332111-                   *          *        *
 30N   1     *        *33CDDDFFC74111-                      *      *        *
 25N 222222211        *    227CHD521    ----* * * *          *        *
 20N 222222211  1 11125346511 *1       -- ---      *          *        *
 15N 222222221145447CB89A652221211     *  -----     ----      *        *
 10N 2222123224DIGGE9AA33432334211   1*-----------------       *       *
  5N 23223337DIUVMOMU?MEA5321111   --------------------*  ---  *       *
  0* 4832899JRU??6GJMMMNMMBLEC43212  *1-------------------     *       *
  5S HMJ35IG859CF2AHDA5612366B54844311--------------- -----    *-      *
 10S ARH4GZVPISPFFBFH9646311222225743212-----------1 ---       *       *
 15S 364447778BFB8?IEOAB77452177CC764344---------------   -- -         *
 20S GE99BCHIFHJHC745542 3637HKMSQKHB9--------11------       --        *
 25S 487H7A6FDEAB531  1134FCGUXO?NDK?VLF---543-------        *        *
 30S *12411111*121     1 26C9KJDJA8FIJJKIMHHJ-------        *        *        *
 35S  231    *        *  11122131111132569985-----        *          *        *
 40S  *        *        *1 1       * 124232312----      *          *        *
 45S   *      - *        1*         13632      1     *           *         *
 50S         --  *         *        1AB832* * * *             *          *
 55S    *         *         *       114C3432                 *          *
 60S     *         *          *           41               *           *
 65S      *         *           *                        *            *
 70S       *         *             * *                 * *           *
 75S        *          *                * * * * * *2221             *      22112
 80S          *          *                          22331         *       675612
 85S           *          *                                     *          1624

      W...W...W...W...W...W...W...W...W...W...*...E...E...E...E...E...E...E.. E   E
    120 108  96  84  72  60  48  36  24  12   0  12  24  36  48  60  72  84  96

 MISSING PERIODS
2240-0000
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ENCL 2

JULY 1990

UPDATE ON NEXT GENERATION WEATHER RADAR (NEXRAD)

THE JOINT SYSTEM PROGRAM OFFICE (JSPO) HAS CONFIRMED THE COMMITTMENT OF 133 COMMUNICATIONS PORTS ON NEXRAD RADAR PRODUCT GENERATORS (RPGS) FOR CORPS OF ENGINEERS USE. THE FUNDS OF $104K WILL BE TRANSFERRED THIS FISCAL YEAR.

THE TULSA DISTRICT WILL AWARD A CONTRACT TO PROCURE THE PRINCIPAL USER PROCESSOR INTERACTIVE EMULATOR (PUPIE) SOFTWARE BY 1 AUG 90. THE CONTRACT HAS THREE DIFFERENT OPTIONS, THE FIRST (COMRAD) WILL BRING THE DIGITAL DATA FROM THE RPG TO THE DISTRICT PERSONAL COMPUTER, THE SECOND (VUERAD) WILL CONVERT THE DIGITAL DATA AND DISPLAY A GRAPHIC PRODUCT ON THE PC AND THE THIRD (COMRAD 5) WILL SEND RAIN GAUGE DATA BACK TO THE RADAR FOR GROUND TRUTH CALIBRATION.

THE REQUEST FOR PROPOSAL FOR THE TRI-AGENCY COMMUNICATIONS SERVICE FOR NEXRAD WILL BE ISSUED IN THE SEPTEMBER-OCTOBER 1990 TIME-FRAME. CONTRACT AWARD IS SCHEDULED BY 1 APRIL 1991. THE COMMUNICATIONS SERVICE SHOULD BE FULLY OPERATIONAL BY LATE SUMMER OF 1991 FOR INSTALLED NEXRAD RADARS.

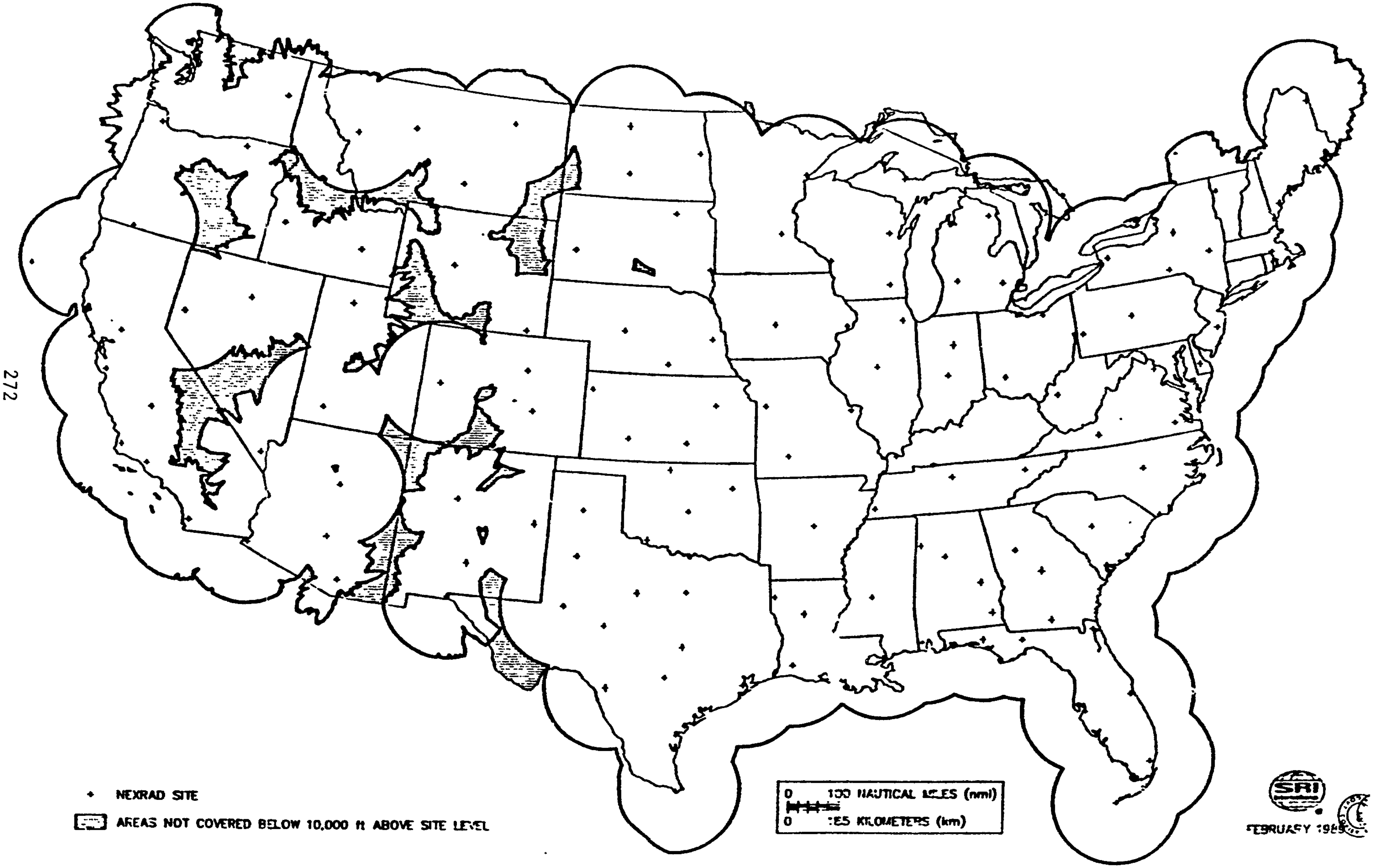
TWO RADAR SITES HAVE BEEN MOVED INTO THE LIMITED PRODUCTION PHASE OF THE NEXRAD CONTRACT, THEY ARE DODGE CITY, KANSAS AND HOUSTON/ GALVESTON, TEXAS. THE CURRENT SCHEDULE FOR EQUIPMENT DELIVERY OF THE LIMITED PRODUCTION PHASE IS LISTED BELOW:

SITE			DELIVERY DATE
NORMAN, OKLAHOMA	OPERATIONAL SUPPORT FACILITY -		INSTALLED
OKLAHOMA CITY, OKLAHOMA	WFO	SWD	MAY 90
MELBOURNE, FLORIDA	WFO	SAD	DEC 90
WASHINGTON, DC (STERLING, VA)	WFO	NAD	FEB 91
FREDERICK, OKLAHOMA	DOD	SWD	MAY 91
EGLIN AFB, FLORIDA	DOD	SAD	JUL 91
ST. LOUIS, MISSOURI	WFO	LMVD	SEP 91
DODGE CITY, KANSAS	WFO	SWD	OCT 91
HOUSTON/GALVESTON, TEXAS	WFO	SWD	NOV 91

FULL OPERATION OF THESE RADARS CAN BE EXPECTED FOUR TO SEVEN MONTHS AFTER THE EQUIPMENT IS DELIVERED.

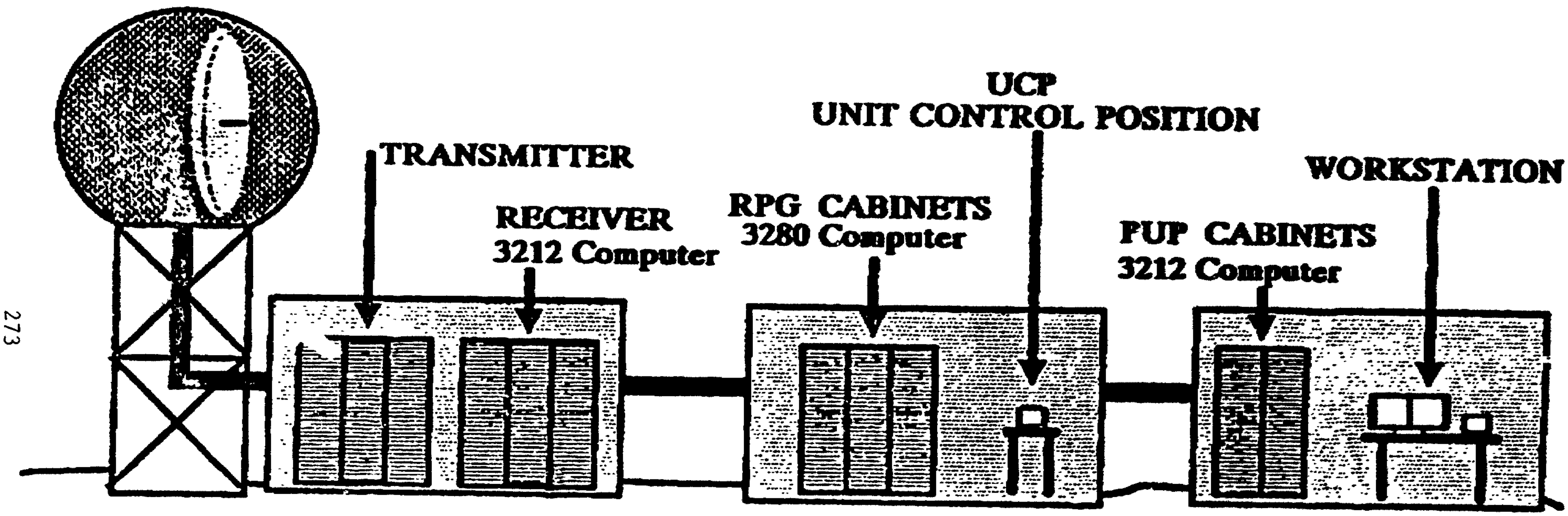
ENCL 3

NEXRAD COVERAGE – COMPLETE SYSTEM



ENCL 4

WSR-88D MAIN COMPONENTS



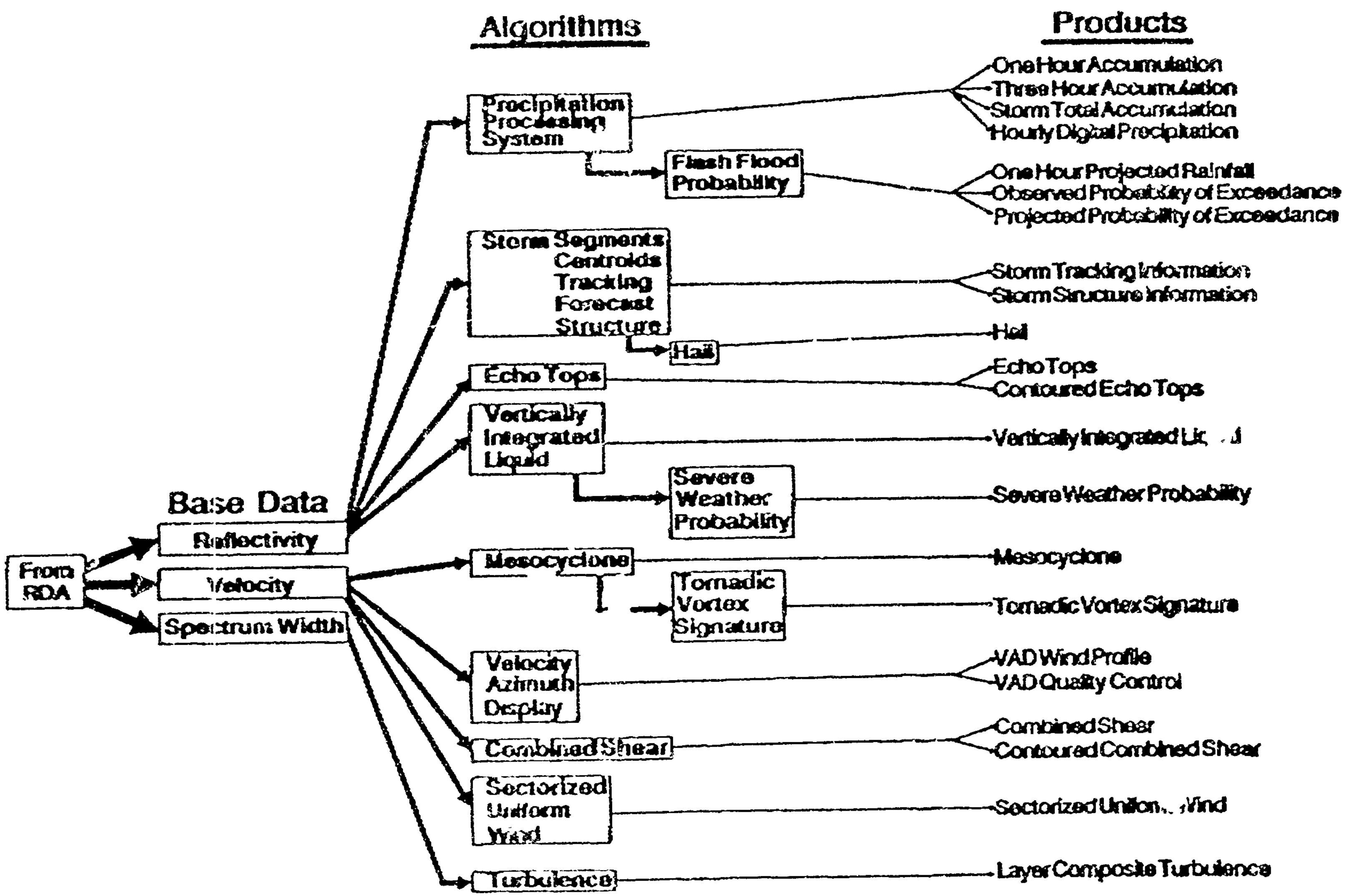
RDA
Radar
Data
Aquisition

RPG
Radar
Product
Generator

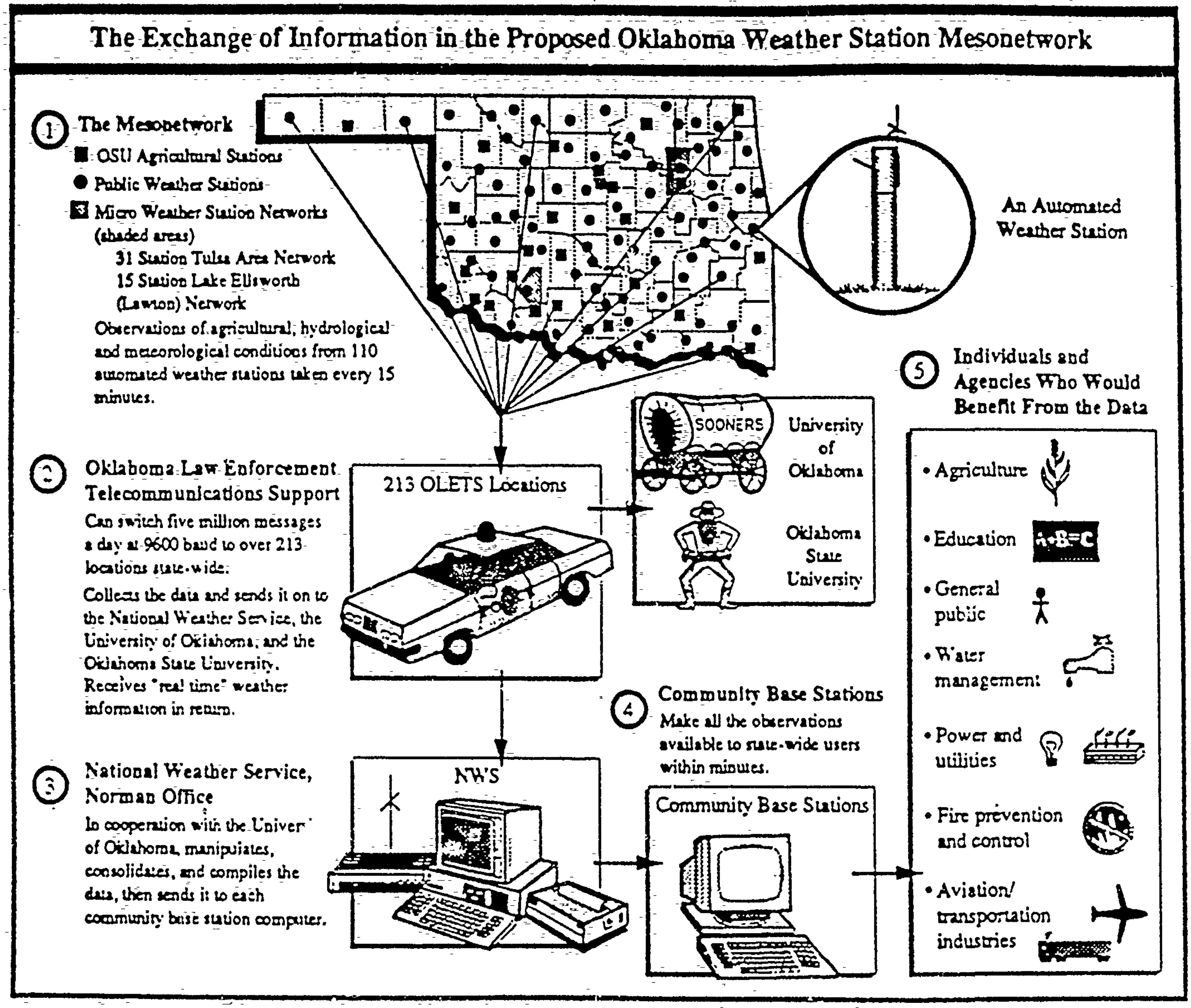
PUP
Principal
User
Processor

ENCL 5

RPG Algorithm Flowchart



ENCL 6



Schematic Design of the Proposed Oklahoma-Wide Mesonetwork of Environmental Monitoring Stations

Radar Applications

by

Carroll E. Scoggins

SUMMARY OF DISCUSSION BY GARY R. DYHOUSE

Q: If we are relying on rain gages to calibrate the radar rainfall estimates, how confident are you the gages are functioning and giving good information? Our experiences are that its very difficult to keep the collector funnel clean and free from debris without very frequent inspections.
A: That has been our experience also. We contract with the USGS for gage maintenance and require them to inspect the gage and clean it on every site visit. We follow up with frequent inspections ourselves.

Q: Why is the USGS doing this instead of the National Weather Service?
A: NWS simply does not have the manpower to do this, where the USGS does.

Q: There may be a problem with the Corps of Engineers being allowed access to NEXRAD ports by the NWS. Will this be overcome?
A: There have been problems between individuals in the two agencies, but this will be rectified at the Washington level. The Corps wants on the NEXRAD system and this is supported by most NWS personnel involved.

Q: Is NEXRAD useful in mountainous terrain?
A: Yes, but not as much as in non-mountainous areas. One of its features is the ability to eliminate false echoes from ground clutter near the radar site. This includes mountains. Mountainous terrain will block reflectivity returns from precipitation, but NEXRAD will still be a powerful improvement. The system cannot detect whether the precipitation is rain or snowfall, however.

Comment: Various studies have indicated that errors in rainfall measurement by on-the-ground raingages may be as much as 20%. We may be relying too much on accurate rainfall measurements at gages to calibrate and evaluate our radar information.

SESSION IV
OPERATIONAL HYDROLOGY

SUMMARY OF SESSION IV
OPERATIONAL HYDROLOGY

prepared by

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Overview

The topics covered in the presentations in this session included selection of the appropriate techniques and equipment necessary for community flood warning, gate operations for two flood control projects, and safety concerns for levees and ringwalls. A panel discussion with short paper presentations was held in the afternoon which addressed planning concerns from a HQUSACE perspective, an update and review on hydropower, comments on water control related to existing projects with supporting case examples, and communication necessary in hydrologic engineering necessary for effective water control. Session IV concluded with a review of the workshop and announcement of the 1991 workshop theme.

Paper Presentations

Mark Nelson, Omaha District, gave a paper entitled "Using Appropriate Flood Warning Technology for Communities at Risk." Mr. Nelson stressed the necessity to select the flood warning system commensurate with the local governments needs and ability to pay. He presented two case studies in the Omaha District. The first study showed that a city had been sold a sophisticated warning system by a commercial vendor. The system never gave a successful warning for a variety of reasons, some of which were lack of backup power and 24 hour staffing. The Corps used the lessons learned from the study of this town to derive a simple, inexpensive, and reliable flood warning plan. This plan had three components which included a burglar/fire alarm, a hydrologic model, and community involvement.

Questions were raised about the patentability of this system. Mr. Nelson replied that an application for a patent had been submitted and that rights had been transferred to the Corps. There were discussions also about use of telephone lines versus radio transmissions. Mr. Nelson responded that phone lines the system used were buried and there had been no problems with receipt of signals. He also stated that the phone lines in use were not dedicated.

Roger Less, Rock Island District, presented a paper entitled "Formulation and Design of Levee Gate Closures, West Des Moines, Iowa." Mr. Less described the joint-effort Alert flood warning system for West Des Moines and Des Moines levee project which involved the two local sponsors, the National Weather Service, and the Rock Island District. An existing flood warning system for the Racoon River with a large drainage area of 3500 square miles was already in place. However with the proposed construction of the Corps levee project giving 100 year flood level of protection, there was concern within the Corps about warning time for making closures along the rapidly-rising Walnut Creek with its smaller drainage area of 82 square miles. Consequently, the Des Moines Alert system was designed to provide real-time and stream level data for Walnut Creek and another minor stream during the interim prior to construction of the Corps levee. A major advantage to this system was that the local government would become highly proficient

with the use of the Alert system during the interim and could readily adapt to a levee system operation upon completion of construction. The system was in place during the June 1990 flood and worked well.

A question was raised about the proper type of closure system for a project. Mr. Less stated that closure structures should logically be based on warning time and as an example, swing gates were appropriate during flash flood situations.

Jim Mazanec, North Central Division, presented a paper entitled "Gate Operations on the Fort Wayne Flood Control Study." The paper centered around the efforts that the Detroit District made to answer questions of workability of road closure structures for the Fort Wayne levee/floodwall upgraded system. The study was made under a variety of historic and hypothetic storm conditions. The existing Alert flood warning system was assumed to be incorporated into the proposed flood control project and was used to estimate warning times. The existing system had already proven valuable in reducing flood damages during previous events but a more extreme event, a 200 year hydrograph, was derived and used as a test at several locations. The test involved defining warning time available and time of road overtopping for each of two rivers based on rates of rise for the 200 year event. The hydrographs were also used to determine warning time available for each river based on the rates of rise and the assumed closure beginning times. These data were compared to the required installation times for the closure structures to determine if sufficient closure times were in fact available. The study concluded that a higher mobilization stage was required since many false alarms would result at the initially selected stage. Available closure times also were insufficient at several other structures. This resulted in a revision of the project to include ramping at four locations.

Further study is planned to determine if the mobilization level is still too low and if sequencing mobilization levels such that the city may take advantage of better managing their work crews. Upstream gages will also be investigated to check to see if additional warning time may be gained.

The initial question raised concerned the performance of the Alert system and its reliability. Mr. Mazanec replied that nothing negative about the system had been received and that the city had a large staff to maintain it. The system was also partially funded by the National Weather Service. Another question was raised about the validity of using 200 year rainfall versus the Standard Project Flood in generating the test hydrographs. Mr. Mazanec felt that the frequency rainfall represented a worse case scenario. Concern was also raised about the ability of the city to make closures since several structures were involved. Mr. Mazanec reiterated that the city had up to 100 people available for emergency duty but that a sequencing study would help to determine how to more efficiently operate the closures.

Larry Holland, Norfolk District, presented a paper entitled "Safety Concerns for Levees and Ringwalls." Mr. Holland's paper centered around use of an Alert System to implement closures for the Buena Vista, Virginia Local Protection Project which provided slightly greater than a 100 year flood level of protection. An additional concern was the ability of the local officials to implement an evacuation plan should the levee be overtopped. His procedure involved developing a test hydrograph composed of critical portions of historical hydrographs to show that sufficient warning time was available. The analysis resulted in showing that the protection system was operable as planned but that much shorter warning times would be available than initially thought. Mr. Holland pointed out the importance of local officials monitoring the river and weather conditions closely during all events and that emergency personnel respond immediately.

A question was raised about "failure to close" as being a HQUSACE standard of operability. Mr. Holland replied in the affirmative, particularly for protected areas that would

have rapid rates of fill if closure were not made. Mr. Holland also agreed that his standard design time should be brought to the attention of the project operator. Lew Smith commented that HQUSACE was reviewing increasingly more projects where the ability to operate the closure structures was questionable. He stressed that strong arguments need to be made to the reviewers on the workability of closures.

Panel Discussions

John Burns, HQUSACE Planning, discussed the topic "Some Planning Considerations." Mr. Burns complimented hydrologic engineers for being involved early in the Planning Process and working with local governments in the planning stages of projects. He stressed that a "sense of reality" needs to be placed into reconnaissance and other initial studies in order to expedite studies and to keep Corps projects affordable to local governments.

Shapur Zanganeh, HQUSACE, addressed the current status of hydropower in the United States. He stated that the Federal Energy Regulatory Commission quotes a figure that only 50 percent of the potential hydropower has been developed in this country. He pointed out that since oil prices are rising again and that the current Iraqi situation in the Middle East is reducing oil availability that hydropower development is becoming increasingly important.

Doug Speers, North Pacific Division (NPD), presented a paper entitled "The Role of Operational Hydrology in Addressing Corps Water Control Issues." Mr. Speers addressed several cases studies in NPD and pointed out pertinent issues. He sees that future work is going to be increasingly more complex and will receive public scrutiny as never before. The public is becoming involved at the early study stages with technical details since they are increasingly concerned with the outcome. The scope of operational studies is much larger than before. He pointed out that guidance at the HQUSACE level was not lacking and FOA's should use it readily. He further stressed that technical tools are available for use in operational studies. Operation and maintenance funds for large studies have not been extremely difficult to obtain but funding for smaller studies is suffering.

Dick DiBuono, Water Control/Quality Section, HQUSACE, presented a paper entitled "Hydrologic Engineering for Effective Water Control Management." Mr. DiBuono stressed that hydrologic engineering during feasibility, design, or reformulation of water resource projects should consider their operability goals to achieve the projects intended purposes. He noted that communication of water control aspects is vital since our local sponsors are responsible for operation and maintenance. For existing projects, communication is becoming increasingly important since the public and elected officials are scrutinizing our water control policies and operations. Mr. DiBuono stated that sufficient information is not transmitted with project reports that are reviewed at his level to show that projects can in fact be operated as designed. One example demonstrated that the project could be operated properly under historic conditions but failed to show that real time decisions could be made during future events.

He also noted by examples of recent droughts and floods that we need to better communicate the effectiveness of our policies and practices in economic as well as hydrologic terms. He also gave an example of the importance of effective water control communications to higher authority which aided greatly in making a national decision. Mr. DiBuono concluded by summarizing that communication of the effectiveness and benefits of our water control plans is vital for gaining public acceptance and Congressional support.

Dick DiBuono, HQUSACE, was questioned about pending legislation which would affect water control operations by requiring that operations plans be strictly followed. Mr. DiBuono thought that this bill would essentially tie the Corps hands in making any operations deviations even during floods or droughts. He thought that some version of the bill would pass although possibly not in its original strong form.

Shap Zanganeh, HQUSACE, responded to a question regarding lack of water quality aspects being addressed in hydropower design. Shap stated that there are some hydropower areas that water quality provisions are very strong. He also stated that there is a hydropower lobby group that wants to dilute the existing laws to promote easier hydropower development.

Doug Speers, NPD, was quizzed about operation of Wynoochee Dam which had been turned over to a local government to operate. Doug responded that the local government essentially had problems operating the project and that the interim solution was that the city would pay the Corps to operate. Roy Huffman, HQUSACE, commented that the Corps still owned the dam and even if the local government were to operate it in the future, that the Corps would be blamed for any mishaps.

John Burns, HQUSACE, was questioned by Roy Huffman on the procedure hydrologic engineers should use to obtain dollars necessary to perform reconnaissance and feasibility studies. John responded that during the recon phase of the study that it was difficult to obtain many dollars since there are limited funds available. However in the feasibility phase, planning formulates a project management plan in which H&H costs can be entered. If there is a dispute, the Deputy Engineer for Project Management has the authority to resolve the differences. John also pointed out that the feasibility level of the studies is cost shared and hydrologic engineers should recognize that costs should be kept as low as possible.

Roy Huffman, HQUSACE, asked Doug Speers if the demand on Columbia River water will exceed the supply in the future. Doug responded that it depends on the annual hydrologic conditions. He stated that currently there is pressure to maintain instream discharges for salmon habitat. He also noted that project operations in the Columbia River basin are under review to insure proper distribution of water during dry years.

Arlen Feldman, HEC, asked a general question to all about their respective districts being asked to cut stream gaging costs. Omaha District feels that gaging costs have already been cut to a minimum. Dick DiBuono, HQUSACE, noted that Dave Wingert, HQUSACE, has already made significant cuts through negotiation with the USGS at the national level. There was feeling that USGS could reduce costs by eliminating publishing data. Dick also pointed out that the Operations and Maintenance people were starting to investigate ways to cut data acquisition costs through a program designated as "Improvement of Maintenance Techniques." He questioned their authority to do so. Roy Huffman noted that Nancy Lopez, USGS, had tried to put together Washington level support for expanding the gaging program but it hasn't gone far.

Carroll Scoggins, Tulsa District, asked a general question about how O&M funds are approved at HQUSACE. Dick DiBuono stated that his office reviews the budget but says that the district's O&M budget officer reprograms budgets many times and H&H has no say in this. HQUSACE is trying to rectify this situation. Lew Smith, HQUSACE, stated that the Assistant

Secretary for Civil Works is getting into the O&M phase of the Corps work and thinks this will help to distribute the funds where needed. He also thinks that we haven't made data collection appealing enough to higher authorities such as districts engineers who could help tremendously in funds allocations.

Lew Smith asked Carroll Scoggins a question regarding his intentions of archiving "NEXRAD" data particularly for use in future hydrologic studies. Carroll responded that he intended to do it.

Review and Summary

Lew Smith, HQUSACE, provided a review of the workshop and discussed the future. Lew stated that he thought that the workshop format was ideal and they should be continued. Specifically, next year the subject for the workshop will be freeboard and will be held in the St. Paul District.

Lew commented that during the first part of the workshop, that the subjects presented really addressed people problems both within and outside the Corps. He noted that the Corps is really not building many more dams and we may not be doing a good enough job with the projects that we have. He thinks that overall Corps projects are facing water shortages but we may use that to our advantage to gain national attention. He stated that the shortages we're facing should be putting demands on our new reservoir projects to help take up the slack.

Lew thought that the second segment of the workshop raised questions of whether our budgeting and decision-making procedures were adequate. He questioned, for example, if the NED plan was really adequate for making national policy decisions since it many times causes projects to end up with low levels of protection. He thinks we need to place a great number of caveats on the NED plan and look for other means of providing higher levels of protection particularly when safety is involved for large urban areas. A whole new scheme for project justification is needed.

Lew sees that we are going in new directions in hydrologic engineering because of new sources of data and because of public involvement. NEXRAD information was given as an example of new sources of data. Paul Rodman's paper, "West Fork Trinity River, Use of Valley Storage," which addresses various scenarios of flood plain fill and its effects on flood heights, was referred to as an example of the new direction of hydrologic studies because of public involvement. New data is also going to require a national decision on archiving.

A final comment Lew made was in terms of "human factors." He pointed out that subjects such as making gate closures, proper use of Alert systems, for example, involve many assumptions about how locals will react in operating the projects the Corps builds for them. He sees hydrologic engineers becoming involved in social sciences to a degree in order to better define human response during flood events.

USING APPROPRIATE FLOOD WARNING TECHNOLOGY FOR COMMUNITIES AT RISK

by

Mark E. Nelson, P.E. [1]

Introduction.

An alternative to sophisticated electronic flood warning systems was needed in order to meet the design requirements of a project in Nebraska. A market search located no systems that would provide the combination of flood warning time, reliability, simplicity and economics that we needed. We then developed a flood warning system to satisfy the project requirements.

The major drawback of the state-of-the-art systems for our application is that they are too difficult for small rural communities to operate and maintain. Yet, many of those towns have a real need for automated flood detection. Additionally, much of the sophistication and speed provided by systems such as ALERT is not required in cases where the basin time of concentration is fairly long. Due to the relatively flat topography of the Great Plains, adequate flood warning can often be provided using stream stage detection alone, without real time rainfall information.

Following the development, testing and operation of the District-developed flood warning system for a Nebraska town, we now are better able to select the proper level of flood warning technology for a recipient community. It is likely that state-of-the-art electronic flood warning systems will still be specified for larger communities in mountainous areas or where the time of concentration is short. Smaller communities located in flat topography, on the other hand, will be able to have flood warning systems that they can afford to operate and maintain.

State-of-the-Art Flood Warning Systems.

Advances in Flood Warning Technology. With the advent of relatively inexpensive powerful microcomputers, flood warning technology made a quantum leap in the past decade. Sophisticated flood warning systems are now available that combine remote rain gage and river stage instruments with powerful software run on base station microcomputers. Those systems have demonstrated

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their worth by saving lives and property across the country and in other nations around the world.

ALERT Flood Warning Systems. The most well known of the computer-based electronic flood warning systems is the ALERT system. ALERT is an acronym that stands for Automated Local Evaluation in Real Time and was coined by the National Weather Service's California-Nevada River Forecast Office (Gimmestad and Barrett, 1986). An ALERT flood warning system consists of a base station or stations and rain and streamflow measuring equipment. A base station usually consists of a microcomputer with peripherals and a radio receiver. ALERT software, written specifically for flood warning, is run on the base station microcomputer, which is usually an IBM PC-AT or compatible.

Information is fed into the base station computer by radio from rain and stream gages located throughout the watershed. Radio transmissions may be aided by repeater stations. The rain gage consists of a tipping bucket-type rain collector, microprocessor, radio transmitter and rechargeable batteries. Stream gages are often built in tandem with a tipping bucket rain gage and sold as combination gages. Appearing similar to the rain gage, the combination gage has additional circuit boards for processing the stream stage data fed to it by a sensor located in the stream. Stream stage sensors include pressure transducers, gas purge manometers and float-stilling well devices. An example of a combination gage is shown in Figure 1.

The ALERT systems are designed to provide flood warning by using a watershed rainfall-runoff model in conjunction with the real-time data supplied by the remote gages to make forecasts of flood stage at downstream locations in the watershed. Rain and stream gages are programmed to increase transmission frequency once threshold values of precipitation and stage have been exceeded. The ALERT software evaluates both the exceedance of the threshold values and rates of change of rainfall and stream stage, then generates downstream stage and discharge forecasts. The predictions are corrected by the arrival of new data and are constantly being updated. This process provides the latest information to community decision makers faced with coordinating a flood fight.

Applications of Sophisticated Flood Warning Systems. The ALERT systems and related systems such as IFLOWS (Integrated Flood Observing and Warning System) have proved to be valuable tools in reducing loss of life and property by providing extra time to take action. There are many success stories since the implementation of modern electronic flood warning technology in the late 1970's. Several of the greatest successes have occurred in California where the technology and methods were developed.

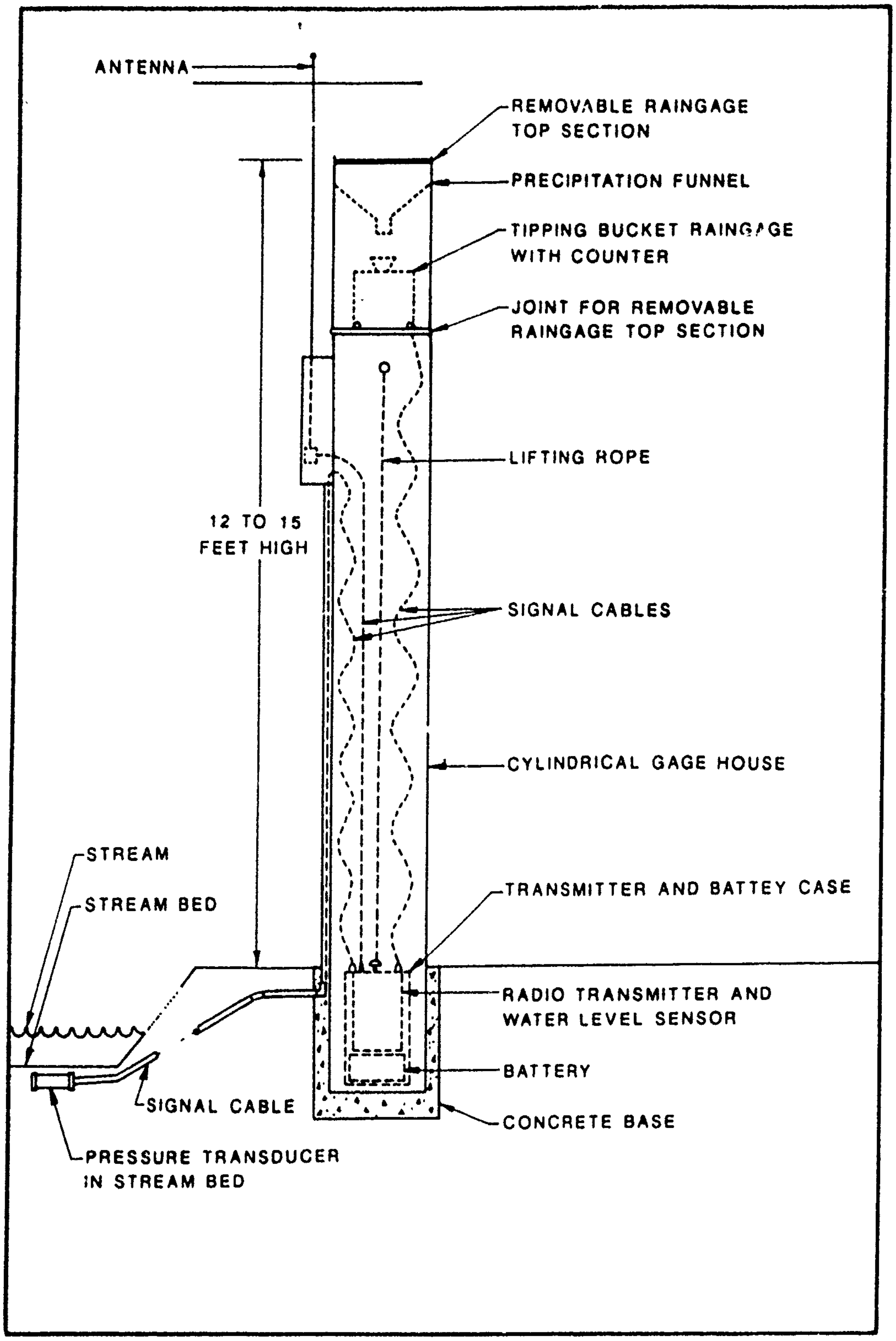

Figure 1.
Typical Alert-Type Combination Gage

Considerable assistance was offered to those California communities by the developers of the ALERT system in the early years (Taylor and Weikel, 1990).

All applications of electronic flood warning have not been success stories, however. Some of the flood warning systems have been placed in small towns or villages that have neither the budget nor the staff to successfully operate, maintain and upgrade the systems over time. Sold by enthusiastic sales people and received by enthusiastic townspeople weary of floods, some of the systems have fallen into disrepair. The Omaha District of the U.S. Army Corps of Engineers had one such experience in the mid-1980s.

An Iowa Community's Experience.

Need for Flood Warning. Following a disastrous flood in 1982, the Corps was requested to build a flood control project for a small town in Southwestern Iowa. A levee was built in 1984 which provides 100-year protection for the town. One road intersecting the levee formed a low spot with inadequate freeboard for the 100-year flood. A few residents, located on the other side of the railroad embankment which forms part of the levee, did not gain flood protection from the project.

Additionally, a heavily traveled road and an Amtrak line cross the stream near the levee. Both are subject to flood inundation. Due to the potential for loss of life on the highway and the railroad, the small portion of the community left unprotected and the freeboard concern where the highway crossed the levee, the need for flood warning became apparent.

Selection of a Flood Warning System. The Omaha District contacted available vendors of flood warning systems in an attempt to buy a system that would provide flood warnings and be easily maintained by the town. Following assurances by company officials that their system would accomplish those goals, the Omaha District purchased an ALERT type flood warning system from one of the vendors. The system included 2 combination gages, 1 precipitation gage, a base station and a pager system. Installation was completed by Corps employees by July 1985.

System Deficiencies. Problems occurred from the start. The training provided by the vendor was inadequate, as described by Corps and local officials in attendance. Only a short lecture was provided, which was too technical for most of the participants. Offers by the vendor to enroll the town maintenance worker in a follow-up training class at company headquarters were not well received by the town, since the cost of sending the employee to the training session would amount to a significant portion of the community's annual budget.

The principal industry in the Iowa town is agri-business. Due to the agricultural depression of the 1980's, the town budget has been very tight. The town has a population of about 500 residents and a staff of two; a clerk and a maintenance worker.

By the time the warranty expired in 1986, numerous equipment failures had already occurred. They included a hard drive failure on the microcomputer base station and 2 instances when the software had to be reloaded by Corps' staff members. Additional problems included pressure transducer failures at both gages, premature gage battery failures, electrical shorts that caused false pager alarms, failures of the base station backup power supply and rain gage funnels that blew away in severe thunderstorms.

Some of the system problems that plagued the town were attributable to the location of the base station. It was located in the town hall which is staffed only during regular working hours and which has no uninterruptable power supply.

This was the best site in the small town. The need for a pager system to alert key residents after working hours, and the lack of a good uninterruptable power supply led to vendor design modifications to fit this circumstance. Those modifications, in particular the backup battery power supply for the base station, added to the system complexity and ultimately decreased its' reliability.

The Eventual Result. For a large city, these problems would have been costly headaches. For this town, they doomed the system. By June 1986, the mayor expressed his frustration for the town by stating "At no time during the year since the installation of the system has it worked properly for any length of time." Town officials and Corps employees were able to keep the system running on a limited basis until the base station hard drive failed again during a power surge in March 1988. The failure of the backup power supply to keep the base station operating during power outages and to shield the hard disk from power spikes, ultimately led to the failure of the entire flood warning system. Lacking money to replace the hard disk, frustrated town officials removed the electronic components from the gages and placed them, along with the base station, into storage in the attic of town hall.

The town is neither unusual or backward, just relatively small in size. The ALERT type systems were developed with larger cities and budgets in mind. Those communities where ALERT systems have worked, typically exceed 20,000 in population and employ professional engineers or highly skilled technicians. Most have an office of city engineer and some municipal building with 24 hour staffing and uninterruptable power. In smaller communities where ALERT systems have worked, long term

involvement by state or federal government has been an important factor. An example would be the IFLOWS system in Appalachia (Barrett, 1986).

Long term involvement by the Corps at the Iowa community was not an option. Under current policy, flood warning systems are considered to be non-structural measures and are subject to the project cost share requirement that the local sponsor pay 25% and the Corps pay 75%. Once the project is turned over to the local sponsor, the Corps is no longer involved with the flood warning system, other than making recommendations during annual project inspections.

A Flood Warning Dilemma.

Additional Projects Requiring Flood Warning. The Iowa town was not the only community in the Omaha District where a flood warning system was proposed as part of a flood control project. Two towns in Nebraska, one in Colorado and another in Iowa were scheduled to receive flood warning systems. All are small to medium-sized towns with limited tax bases. It was evident that they would face problems with state-of-the-art flood warning systems, similar to those previously encountered at the Iowa community.

The Next Project. The first of these projects advanced to construction was at a Nebraska farming community of about 1000 people. The Nebraska town, though twice the size of the Iowa community, shares many of the same economic constraints. The paid town staff consists of a clerk and two maintenance workers. Those people, like their Iowa counterparts are innovative and hard working employees, but they lack the training in electronics and computers needed to operate and maintain a state-of-the-art flood warning system.

The flood control project consists of a levee surrounding the town to protect it from floods from a nearby stream. A closure structure across the main highway that runs through the town was necessary to complete the flood protection. Due to the flash flood potential, effectively demonstrated in a damaging flood in June 1984, a flood warning system was required so that the city would have enough time to close the levee. A computer-based electronic flood warning system had originally been proposed for the Nebraska town, but given the earlier experience, that type of system was no longer considered to be a viable option.

Appropriate Technology Not Available on the Market. Flood warning system vendors were contacted again. It soon became evident that all of them were marketing systems that were beyond the capability of most small communities to operate and maintain.

The only option available was to develop a new flood warning system that would meet those needs.

A Solution for the Nebraska Community.

Necessity - The Mother of Invention. Fresh from the Omaha District's experience in Iowa, there were several objectives in mind as development began on an alternative flood warning system. First among the objectives was that the flood warning solution had to be at an appropriate technology level. Once turned over to the small community, it had to be affordable, durable and easy to operate and maintain. Low initial cost and local availability of spare parts were also important objectives. The design that evolved from those objectives featured a combination of equipment, a hydrologic model of the stream and community involvement.

The Flood Detection Equipment. The equipment is a stage warning device which is simple by comparison to the ALERT stage sensing gage. The stage warning gage consists of a telephone alarm dialer mounted in a shelter atop a stilling well which contains float switches mounted on a vertical rod. The telephone alarm dialer plays prerecorded flood warning messages to individuals designated to receive them. The alarm dialer is activated by the float switches.

Two separate message channels are available on the alarm dialer; one for burglary and one for fire. The lower float is attached to the burglary circuit and the upper float to the fire circuit. The use of two floats, generating separate warning messages, permits a rate of rise to be determined. The computed rate of rise can be compared with rates of rise, characteristic of approaching floods as defined in the hydrologic model. The middle float opens the lower float circuit enabling the upper float alarm to transmit when water reaches the upper level.

Rechargeable batteries power the alarm dialer. Buried phone lines carry the warning messages to preassigned city and law enforcement personnel via the local phone system. A staff gage, mounted near the stilling well, provides a visual confirmation of the stream stage and can be used in between alarm transmissions to estimate the rate of rise. A drawing of the stage warning gage is shown in Figure 2.

The Basin Hydrologic Model. The hydrologic model, that was used in the flood control project design, was used to analyze flood warning time. With the model, it was determined that 2 stage warning gages in the basin would provide adequate warning time to close the levee and that automated rainfall detectors were not necessary. The hydrologic model was also used to develop characteristic hydrographs for the watershed using

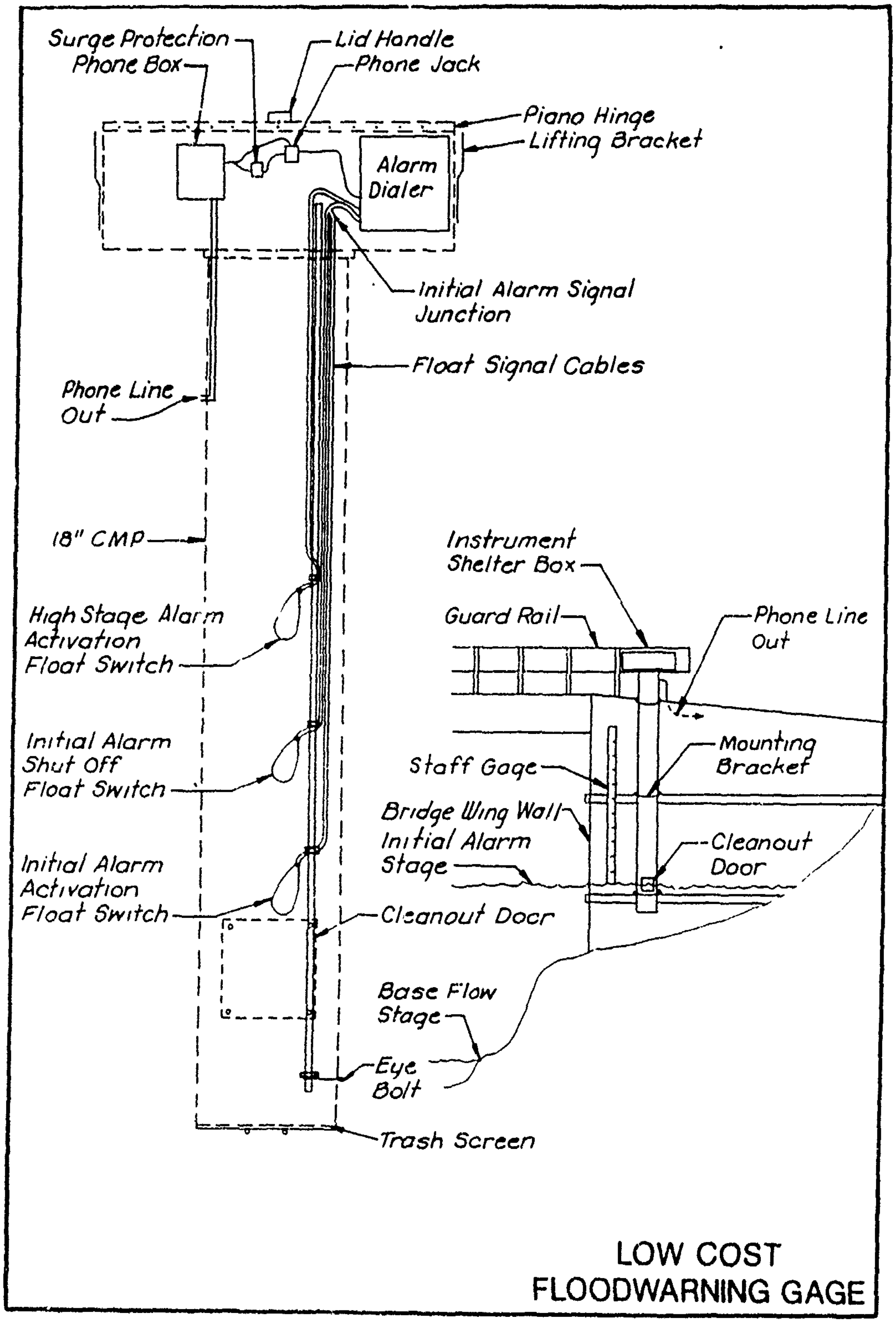

Figure 2.
Stage Warning Gage

different storm intensities and orientations. The hydrographs were then used to define critical rates of rise that would be used to identify the severity of the approaching flood. The rate of rise information was then consolidated into simple lookup tables which were listed in the project operation and maintenance manual. No computer is required to operate the flood warning system.

Community Involvement. Although mentioned last, the community involvement component is extremely important to the long term success of the flood warning system. Community involvement includes the flood response plan and a plan to test, operate and maintain the system. The plan for the Nebraska community was developed by all parties involved and was included as a chapter in the overall project operation and maintenance manual. Community involvement should start early in the project development phase and involve the community leaders, those that will maintain the system and county and state officials involved in civil defense. Without community acceptance, the flood warning system will likely not provide the level of protection intended by the designer.

In this case the National Weather Service's Omaha office also became interested in the system. The Weather Service and the town agreed to share information during floods. The town will call the Weather Service and report high water and will in return receive a radar-based rainfall forecast.

The Prototype. A prototype flood warning system was built by employees of the Omaha District and the local sponsor in the Spring of 1989. All parts for the system were purchased from suppliers in the Omaha area. The flood control project was scheduled for completion by Spring 1990, which allowed for nearly a year of testing and development before the town had to assume financial responsibility for the flood warning system. During the summer, an operation and maintenance manual was drafted and was reviewed by the town employees. The manual defined the communities flood response plan to the warnings and outlined procedures for flood warning drills.

From the start the two town maintenance workers from the town played an active role in the installation and testing of the prototype system. They also helped to install it and suggested design modifications, which resulted in improvements. By late Spring 1989, the prototype system was on line awaiting its first test.

The First Test. The test came early on the morning of September 7, 1989. That morning, very heavy rains crossed the Northern third of the watershed. By 0520, flood waters engaged the lower float of the upstream flood warning gage. City

employees were called by the alarm dialer. They and the mayor drove out to look at the advancing flood waters. With water well below bank full at the downstream gage, they proceeded to the upstream gage. They arrived at the upstream gage by 0600, noticed that the stream was still rising toward bank full and contacted the Sheriff's office. Low roads were closed by deputies before flood water inundated the intersection West of the gage. Since the heaviest rain was concentrated in the upper part of the watershed, the flood attenuated and the flow remained below the low float of the downstream gage and there was no threat to the levee opening.

As a result of that timely test, modifications were made to both the equipment and the community response plan. On October 19, 1989, the flood warning system became fully operational. Following public meetings with federal, state and local officials, the operation, maintenance and testing plan was finalized and delivered to the community in March 1990.

Some Lessons Learned From The Two Systems.

The Omaha District's experience with both warning systems has provided it with many lessons. Among those is an appreciation of when automated flood warning systems should be used, and when they shouldn't, the need to thoroughly understand the community needs before designing a system and to use technology that can be understood by all of the people involved.

When Not To Use a State-of-the-Art System. When it is evident that a microcomputer-based flood warning system is beyond the capabilities of a community, a less sophisticated method of flood detection should be employed. Factors which should signal the designer not to use a system requiring a computer base station include:

1) Absence of a facility with an uninterruptable power supply and 24 hour per day staffing.

2) Lack of city staff capable of maintaining the base station computer and software.

3) Lack of an adequate budget for continued operation, maintenance and eventual replacement of expensive critical components.

4) Lack of community support for a complex system.

Maintenance and Replacement is Forever. The biggest drawback to automated flood warning systems is the need for long-term maintenance and testing. Any flood detection system, whether it is a sophisticated ALERT network or a low cost system

like the one developed by the Omaha District, requires maintenance. That maintenance, and eventual replacement, must be carried out indefinitely in order to preserve the increment of public safety gained by installing an automated flood detection system. Periodic drills must be conducted to ensure that the system is operational and that citizens know how to respond to the warnings. If a community can't commit the resources to support the system over a long period of time then automated flood detection is not an answer to that communities flood problems.

<u>When Not to Use Any Flood Warning System</u>. While the availability of automated flood warning systems of all kinds has provided an important new tool in reducing flood hazards, it should not be used indiscriminately. Several circumstances exist when a flood warning system should not be specified for a community. They would include:

1) When a cost effective structural or zoning solution exists that won't require citizen action during a flood and long term detailed technical maintenance to guarantee its operation.

2) When a flood warning system would not provide a significant amount of additional time to effectively save lives or property.

3) When a community doesn't want a system or doesn't have enough interest to help prepare an emergency response plan.

4) When the community is not willing to accept the financial burden or to develop a long-term operation maintenance and replacement plan to keep the system functioning.

<u>Conclusion</u>.

Flood warning systems are an important addition to the options available in reducing flood threats to lives and property. The process for selecting equipment and designing the system must be subjected to the same methodology as other engineering decisions. The flood warning system must prove itself cost effective, durable and appropriate for the recipient community. It is vital that the community be actively involved in the design and development of the flood warning system from the outset. An objective of the designer should be to make the community feel ownership of their system by the time that the project is transferred to their control. Given an active partnership between design engineers and the local sponsor, automated flood warning systems can play an increasingly more important role in saving lives and protecting property in communities at risk.

References Cited:

Barrett, C. B. (1986) "Local Flood Warning Systems" Professional paper prepared for HEC seminar held in Asilomar Pacific Grove, California.

Gimmestad, B. and Barrett, C. (1986) "The Alert Handbook" NOAA, National Weather Service.

Taylor, D. B. and Weikel, J. G. (1990) "Ventura County Alert System - A Case Study." Professional paper prepared for HEC training course; Flood Warning Preparedness Programs held in Sacramento, Ca.

FORT WAYNE, INDIANA - LEVEE CLOSURE TIMES

by
Darryl Dolanski[1]

Introduction

Based on a review of the Draft Feasibility Report for the Fort Wayne Flood Control Study, in December 1987, by the Board of Engineers for Rivers and Harbors (BERH), OCE, and North Central Division (NCD), the Detroit District was instructed to evaluate the workability of road closures under a variety of flood events. Consequently, the District formulated a methodical approach for analyzing the adequacy of road closure times for the proposed project.

Physical Setting

The City of Fort Wayne, Indiana, is located in northeastern Indiana in Allen County, and is part of the Maumee River basin, which drains into Lake Erie at Toledo, Ohio. The city is located at the junction where the St. Marys and St. Joseph Rivers join to form the Maumee River. Major floods occur when high flows are experienced in both rivers at the same time.

The Flood Control Study for Fort Wayne and vicinity was authorized by Congress in 1972. The authorization requested the Corps to determine the advisability of providing improvements for flood control and allied purposes at and in the vicinity of Fort Wayne.

Alternative plans which were analyzed in the Feasibility study included several plans to divert flood flows via a Trier Ditch cutoff channel, which would run southeast of the city; and several plans for reconstruction of the existing levees and floodwalls with either evacuation or new levees/floodwalls for other areas.

The recommended plan is to upgrade 35,000 feet of existing levees and floodwalls along the Maumee, St. Marys and St. Joseph Rivers, and Spy Run Creek. The levees/floodwalls would be improved to provide a 100-year level of protection, plus freeboard.

[1]Hydraulic Engineer, Detroit District, U.S. Army Corps of Engineers

The levee/floodwall system would cross 24 bridge approaches, as well as crossing two streets. Based on the fact that significant backwater does not occur at any of the bridges, they would be left in place under the recommended plan. Nine of the approaches have high points which would exceed the top of levee, so no closure would be necessary. Two of the approaches and both street crossings have high points which would fall below the top of levee, but would exceed the 100-year design level. Closure of these approaches would be accomplished with sandbags or a temporary clay dike. The remaining thirteen bridge approaches would require a stoplog structure to maintain the integrity of the levee system.

A review of the Draft Feasibility Report by the Board (BERH), OCE and NCD indicated a need to evaluate the workability of the closures under a variety of flood events, both historical and hypothetical. The flood warning system currently in place in Fort Wayne would be integrated into the Federal project and used to estimate flood warning time to aid in accomplishing the necessary closures. This system, acronymed "ALERT"(Automated Local Evaluation in Real Time), was purchased by Fort Wayne following the 1982 flood. The system interrogates tipping bucket rain gages for precipitation data, water stage recorders for river levels, and a thermocouple for air temperature. Collection of these parameters is accomplished through radio links with a microcomputer located in the National Weather Service (NWS) office in Fort Wayne.

This system was in operation during the February 1985 flood, which resulted from a rainfall/snowmelt event, and was the third highest flood of record on the Maumee River. City and NWS officials indicated that the flood alert system enabled a prediction to be made that the Maumee River would exceed flood stage three days before flood stage was reached and eight days before the river crested. This allowed sufficient time to mobilize flood fighting efforts and would have allowed ample time to install the road closure structures, had the Federal project been in place. Also, the crest elevation for the Maumee River was predicted, by virtue of this system, to within 0.1 foot. In short, the flood alert system was credited with substantially reducing damages during the 1985 flood.

A more strict test of the flood alert system's ability to allow sufficient time for road closures to be effected would occur during a flood caused by intense rainfall and involving a sudden rise in river stages. A minor flood of this type occurred in June 1981, when three inches of rain fell in half a day. The Maumee River rose at the rate of one foot per hour and exceeded flood stage by four feet. Road openings in the proposed levee system would not have been affected by the June 1981 flood, however, since that flood was approximately a 5-year event and the existing bridges are at higher elevations.

Study Approach

A hypothetical rainfall/runoff event, similar to but more severe than the June 1981 event, was modelled using HEC-1 to test the workability of the road closure structures under a sudden and large rise in river stage. A 200-year runoff event was selected and modelled by applying the appropriate 12-hour duration rainfall to the St. Marys and St. Joseph Rivers' unit hydrographs to obtain flood hydrographs. The two hydrographs were then combined to obtain a flood hydrograph representing the Maumee River at Fort Wayne. These flood hydrographs were used in conjunction with HEC-2 rating curves, developed for each road requiring a closure structure, to determine the time when road overtopping would occur. The hydrographs were also used to determine the warning time available on each river based on the rates of rise for the assumed event and the assumed mobilization stage for each river; that is, when road closings would commence. This information was evaluated against the required installation time for each closure structure to determine if sufficient time would be available for closures to occur, based on the assumed scenario and allowing for a factor of safety. This analysis has been summarized in table format.

Study Results

The following observations warrant discussion regarding specific aspects of this table:

1. For this analysis, mobilization stages were assumed to be the official flood stages designated for the ALERT stream gages in Fort Wayne, and the table's Alert time was assumed to occur one hour before mobilization time.

2. Mobilization and road overtopping times were taken directly from the HEC-1 200-year flood hydrographs.

3. Estimated closure times were obtained based on discussions with Detroit District's Emergency Management Branch, NCD, and St. Paul District. It is assumed that all materials needed for a closure will be on-site.

Conclusions

The following conclusions were reached based on the analysis as summarized in the table:

1. Basing Maumee River mobilization on the NWS gage flood stage would result in many false alarms due to the small recurrence interval of flood stage (1.3 years) on that river. A higher mobilization stage would be more realistic based on the large safety factors on that river.

2. Available closure time would be inadequate for the two Junk Ditch closures, and two of the three Spy Run Creek closures, based on their low safety factors.

As a direct result of the levee closure analysis, the levee/floodwall plan was revised to include ramping of the four roads which had closure time safety factors which were less than three. These roads would be ramped up to the elevation which results in a safety factor of three.

The levee closure analysis was conservative in that only stream gages in Fort Wayne were used to determine alert and mobilization times. This restriction assumes that only local flooding is occurring which would not be detected by upstream gages. Under this assumption, adequate warning time would not be available for the closure structures on Junk Ditch and Spy Run Creek, as previously discussed. However, the Fort Wayne flood alert system includes upstream gages on the St. Joseph River near Newville, Indiana and the St. Marys River at Decatur, Indiana. The National Weather Service has estimated that an additional twenty four hours of warning time would be available on the St. Joseph River and eighteen hours on the St. Marys River based on travel times on these rivers. Therefore, it is reasonable to assume that as much as one day of additional time would be available to accomplish the road closures under a basin-wide severe rainfall event.

The City's flood alert system also provides advance warning whenever a severe localized rainfall occurs regardless of river stages. Under this scenario, a warning alarm sounds dependent upon precipitation amount and antecedent soil moisture conditions.

It should be noted that this analysis is currently undergoing modification and refinement in the Design Phase of this study. It is considered a dynamic analysis which will undergo various levels of sophistication, and will ultimately be utilized to develop an operations manual for the project.

One change which is foreseen, for example, is a lowering of the safety factor (which corresponds to an increase in the mobilization time) at several locations. If a safety factor of 2 is considered to be adequate, it is much too high in several areas. The problem with too high of a safety factor is that it causes mobilization to occur sooner than it really needs to. Thus if the available closure time is high, it might be worthwhile to delay mobilizing, since there is a chance that the stages might subside before that time. An extra bonus of lowering these factors is that it might result in a staggering of the mobilization times, which in turn could assist the city in better managing their work crews during a flooding situation.

Another revision which may warrant consideration is inclusion of the upstream gages (at Newville and at Decatur) into this analysis, which could result in additional warning time to accomplish road closures. However, before such a revision is made, a better understanding of the relevance, to the Fort Wayne area, of the data collected at these gages is needed. Although it may be beneficial to incorporate precipitation gages into this analysis, the extensive network of existing ALERT stream gages in the area (7 plus 1 new gage planned) may preclude the need for analyzing flood warning time based on precipitation data.

In conclusion, the importance of this fundamental, yet germane application to the flood control arena should be recognized and integrated into the planning process.

SUMMARY OF ROAD CLOSURE STRUCTURE ANALYSIS

CLOSURE STRUCTURE LOCATION	TYPE[a]	CLOSURE DIMENSION (H x W,FT)	ALERT[b] STAGE (FT)	ALERT[b] Q (CFS)	ALERT[b] T[d] (YR)	ALERT[b] TIME[e] (HR)	MOBILIZATION[c] STAGE (FT)	MOBILIZATION[c] Q (CFS)	MOBILIZATION[c] T[d] (YR)	MOBILIZATION[c] TIME[e] (HR)	ROAD OVERTOPPING STAGE (FT)	ROAD OVERTOPPING Q (CFS)	ROAD OVERTOPPING T[d] (YR)	ROAD OVERTOPPING TIME[e] (HR)	ESTIMATED CLOSURE TIME[f] (HR)	AVAILABLE CLOSURE TIME[g] (HR)	SAFETY FACTOR[h]
MAUMEE RIVER																	
TECUMSEH	SL	7.4 X 60	746.0	8000	1.1	12	746.8	9200	1.3	13	754	21000	15	21	2	8	4.0
COLUMBIA	CD	2.9 X 70	746.6	8000	1.1	12	747.4	9200	1.3	13	760	32000	280	38	1.5	25	16.7
ST MARYS R.																	
FOURTH	SL	5.8 X 60	749.9	6000	2.0	11	750.9	6900	2.5	12	757	14300	30	18	2	6	3.0
WELLS	SL	3.2 X 70	750.1	6000	2.0	11	751.1	6900	2.5	12	760	18000	110	23	2	11	5.5
VAN BUREN	SL	4.4 X 80	750.4	6000	2.0	11	751.4	6900	2.5	12	760	17200	80	22	2	10	5.0
MAIN	SL	6.1 X 80	750.6	6000	2.0	11	751.6	6900	2.5	12	759.2	15800	50	20	2	8	4.0
ST JOSEPH R.																	
TENNESSEE	SL	5.7 X 80	752.4	9000	4.5	19	753.2	10000	6.0	20	758	17400	70	28	2	8	4.0
STATE	SL	4.2 X 80	752.8	9000	4.5	19	753.5	10000	6.0	20	760	19800	140	31	2	11	5.5
PARNELL	CD	3.1 X 60	754.5	9000	4.5	19	755.3	10000	6.0	20	764	22700	330	40	1.5	20	13.3
ADDISON	CD	3.7 X 50	755.0	9000	4.5	19	755.9	10000	6.0	20	764	21600	250	36	1	16	16.0
SPY RUN CK																	
FOURTH	SL	6.7 X 60	749.9	6000	2.0	11	750.9	6900	2.5	12	756	13100	20	17	2	5	2.5
ELIZABETH	SL	8.7 X 40	749.9	6000	2.0	11	750.9	6900	2.5	12	754	10500	8	14	2	2	1.0
STATE	CD	3.2 X 60	749.9	6000	2.0	11	750.9	6900	2.5	12	760	18400	125	24	1.5	12	8.0
JUNK DITCH																	
GREENWOOD	SL	13.9 X 50	750.6	6000	2.0	11	751.6	6900	2.5	12	752	7000	2.5	12	2	0	0
EDGARTON	SL	9.9 X 40	750.6	6000	2.0	11	751.6	6900	2.5	12	756	11400	12	15	2	3	1.5

[a] SL=stoplog, CD=clay dike.
[b] Alert status assumed to occur one hour before mobilization.
[c] Mobilization assumed to occur at official flood stage of ALERT gage as follows: (1) Maumee River-745.07 at NWS gage near Anthony Blvd,; (2) St. Marys River-762.97 at USGS gage near Anthony Blvd. extension; (3) St. Joseph River-765 at USGS gage 1.3 miles upstream of Mayhew Road.
[d] T=Recurrence interval.
[e] Time=Time of occurrence based on assumed hypothetical flood scenario.
[f] From Detroit District, Emergency Management Branch.
[g] Time at overtopping minus time at mobilization.
[h] Available closure time divided by estimated closure time.

FORMULATION AND DESIGN OF LEVEE GATE CLOSURES
WEST DES MOINES, IOWA

by

Roger A. Less, P.E.[1]

Introduction

The Rock Island District along with other U.S. Army Corps of Engineers districts have been involved in formulating, designing, and constructing local flood protection projects since the Flood Control Act of 1936. One of the primary methods of providing protection to an area is by containing riverine floodwaters via levees. The use of levees to protect flood plain lands is an ancient technique. A levee is simply a continuous ridge of earth constructed above existing flood plain topography. In earlier times, existing development was rather easily relocated to conform to a new ridge providing levee protection. However, in today's world, development is not so easily relocated. Ramping streets, highways, and railroads over a levee must take into account prevailing speed limits, sight distances, maximum allowable gradient changes, bridge approaches, and many other factors and the construction costs thereof.

In many projects, the decision must be made to make the levee discontinuous due to breaks to allow for existing development that is either infeasible or too costly to relocate. These breaks make for closure structures that must be closed in times of flooding in order for the levee to be made continuous and thus serve its purpose. The ability to make timely closures is critical for a functional and safe levee flood protection project. The decisions regarding closure locations and abilities start in the project's plan formulation and continue through its design and operation.

The West Des Moines - Des Moines, Iowa, local flood protection project includes levee gate closure structures on two streams, one with ample advance warning time and one with limited advance warning time. The plan formulation and design of the overall project included the overriding operational parameter of the advance warning time available at each potential gate closure location. This approach has resulted in a functional flood protection project that the cities of West Des Moines and Des Moines can easily and safely operate.

Physical Setting and Background Information

The West Des Moines - Des Moines, Iowa, local flood protection project is an ongoing project in the Rock Island District scheduled for construction in FY 1992-1993. The cities of West Des Moines and Des Moines comprise the major portion of the central Iowa Des Moines Standard Metropolitan Area (SMA). The Des Moines SMA had an estimated 1990 population of 389,800 with a city of Des Moines population of 196,700 and a West Des Moines population of 29,000. The project area is subject to flooding from the Raccoon River, a major 3,525 square-mile tributary stream of the Des Moines River, and Walnut Creek, an 82 square-mile tributary stream to the Raccoon River. The basins are shown on Figure 1. The Des Moines River is located approximately seven miles downstream from the project site.

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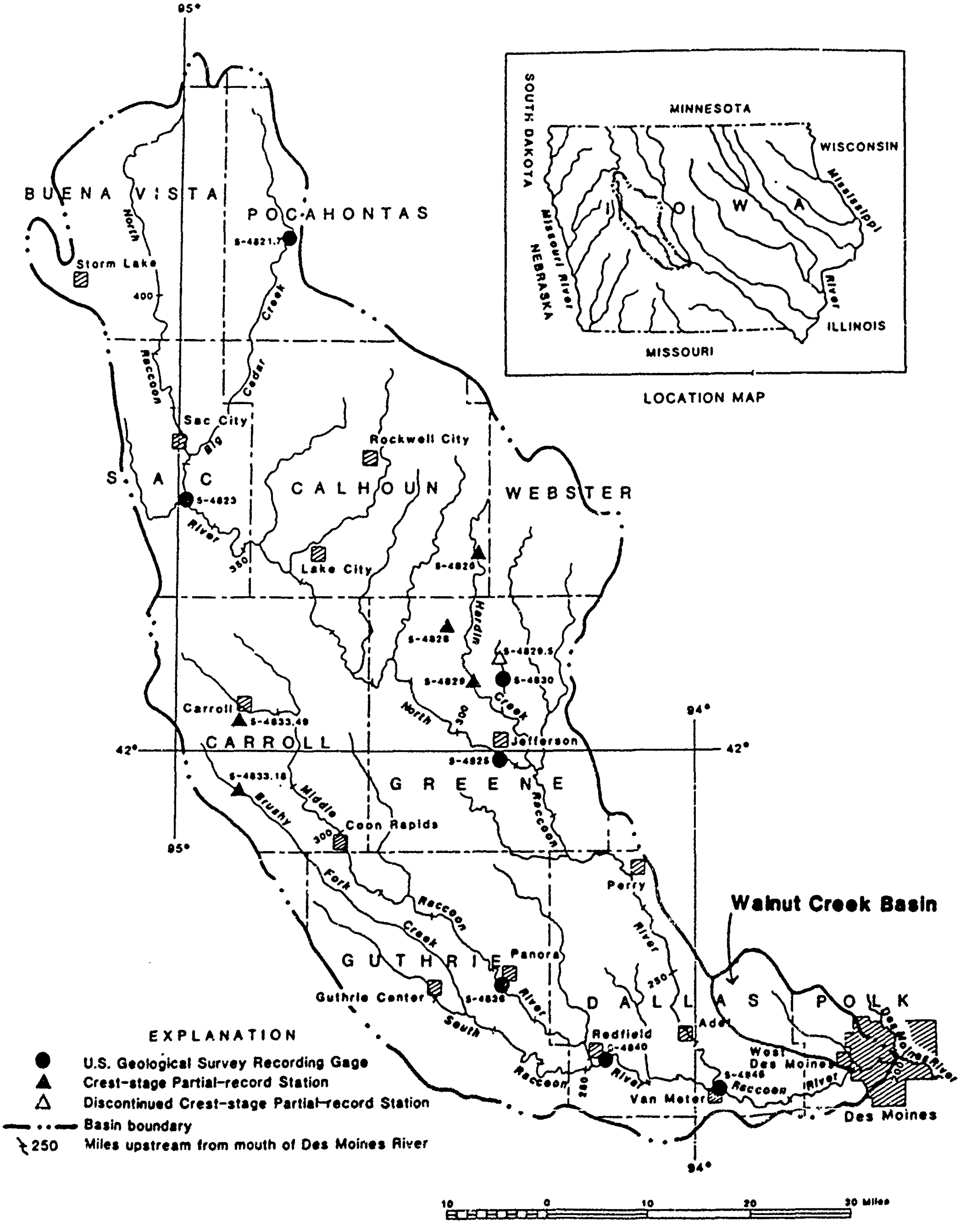
[1] Hydraulic Engineer, Rock Island District, U.S. Army corps of Engineers



Raccoon River Basin

Figure 1

The West Des Moines - Des Moines project provides a 100-year level of protection plus freeboard from Raccoon River and Walnut Creek flooding. Flood protection will be provided to approximately 900 flood-prone urban acres that include 904 residences, 227 businesses, and 11 public buildings. Included in this area is the historic downtown area of West Des Moines, locally referred to as Valley Junction. The project consists of 3.9 miles of earthern levee, 1900 feet of concrete floodwall, 7 mechanical gate closure structures, 2 sandbag levee closures, 3 road ramps, 2 pump stations, 10 interior stormwater gatewell outlets, and 4 ponding areas. The project layout is shown on Figure 2.

Four of the gate closure structures and seven of the interior drainage gatewells are affected by the Raccoon River, a flood source that provides ample advance warning time. The remaining five gate closures and three interior drainage gatewells are affected primarily by Walnut Creek, a flash flood stream. Thus, limited advance flood warning time is available on Walnut Creek.

The existing emergency response measures to a flood situation are dictated by the advance warning time available to the cities. Presently, with no comprehensive levee project for this area, the locals' actions are limited to flood-fighting activities on the Raccoon River, such as placing sandbags and temporary earth berms to hold back intermediate-level floods. This was successfully done during the July 1986 flood due to an advance warning. On Walnut Creek, flash flooding limits local responses mainly to evacuations and utility shut-offs. Even these limited safety and damage reduction measures were severely constrained by a lack of information during the May 1986 flood on Walnut Creek.

Study Approach

Formulation. Both the cities of West Des Moines and Des Moines have well- staffed and trained public works, engineering, police, and fire departments. However, even the most well-equipped city's resources can be strained by a rapidly developing, middle of the night, weekend flood. Thus, gate closures for the project were formulated and designed for the minimum number necessary for a workable, cost effective project.

Levee gate closures were formulated based on descending criteria of:

a. Eliminate a potential closure, if possible;
b. Raise the level of closure, if feasible;
c. Provide the best plan for operability of closure.

The elimination of a potential road closure structure can be done by ramping the road over the levee. This was done at the Valley Drive levee location where the city of Des Moines and the Rock Island District coordinated the Walnut Creek bridge replacement project in the 1980's to include a road raise such that a closure structure would not be necessary. Road ramping is also recommended at two other locations, Winona Drive and Lincoln Road.

The second criterion is to raise the level of closure to the highest feasible level. Several Walnut Creek alignments were studied, which involved various road closures, each with varying levels of closure. The recommended alignment locates the Grand Avenue and 63rd Street Walnut Creek closures at the highest level possible in the existing roadways. The levels of these roadways could not feasibly be raised due to the intensely developed nature of the 63rd Street and Grand Avenue area. To raise roads would involve bridge replacements, railroad embankment raises, arterial street raises, and all the accompanying local access problems. Additionally, the hydraulic characteristics of the area would have been negatively impacted by any road and railroad raises.

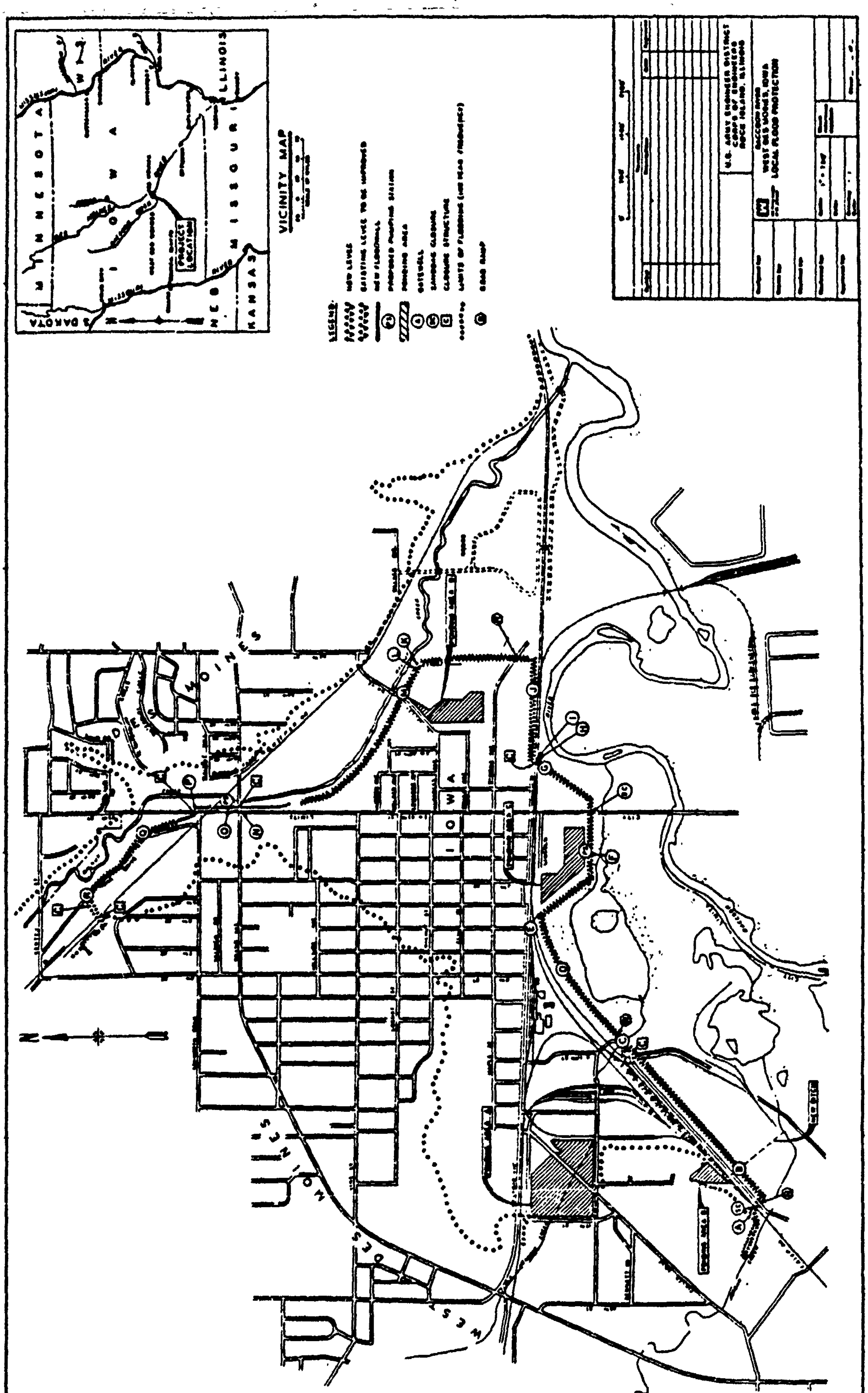

VICINITY MAP
MINNESOTA
IOWA
MISSOURI
ILLINOIS
KANSAS
S DAKOTA
NEB
LEGEND:
DES MOINES
IOWA
WEST DES MOINES

Figure 2

On the Raccoon River portion, coordination with the Iowa Department of Transportation's ongoing 63rd Street improvements has resulted in the 63rd Street closure being in the freeboard range.

The third levee gate closure formulation criterion is to provide the best plan for operability of the closure. To accomplish this, only closures located in the freeboard range are sandbag-type closures. Closures located below the design flood level are swing- or slide-gate structures, which can be efficiently closed by the locals in a timely manner. The railroad and remaining road closure structures fit under this third criterion. The railroads could not feasibly be raised for the same reasons as listed for the roads. Thus, the on-site mechanical type gates are recommended. Actual closure times of one hour on the railroad and two-lane road closures and 1.5 hours on the road closures of more than two lanes have been established.

Interior drainage facilities also need closure during periods of flooding. Again, the minimum number necessary for a workable, cost effective project was formulated. Numerous interior drainage gatewell outlets increase the chances of one being overlooked and not closed when responding to possible rapid nighttime operation. The ten interior outlets, seven on the Raccoon River and three on Walnut Creek reflect distinct interior drainage areas and yield an operable number for each city.

The overall number of levee and interior drainage closures, as well as the sequential operation of the closures, must be in an acceptable range. Ten phased closures can be a workable number. However, ten closures requiring closure all at the same elevation is possibly unworkable. As later tables will show, the West Des Moines - Des Moines project results in a workable phased schedule of closure elevations. Additionally, primary responsibilities for closure are equally divided between each city with the other city having backup responsibility. Providing the best plan for operability also includes providing an adequate advance warning via a flood warning system.

Flood Warning System. The West Des Moines - Des Moines project will be fully operational only if advance warning of impending flooding is made available and put to use. Raccoon River flood warning and response is typical of most major streams where days of advance warning time is available. Flood-fighting procedures to guard against Raccoon River flooding have been institutionalized in each city and the new project operations will be included in their respective flood emergency plans. Therefore, Raccoon River flooding will not be expanded upon.

The cities of West Des Moines, Des Moines, and Clive recognized the need for timely and accurate Walnut Creek flood warning information as a result of the May 1986 flood. Additional advance warning time would allow for crucial additional response time. Evacuations could be made before the onset of flooding, individual flood-fighting measures could be put in place, and emergency services could be alerted and placed on standby.

In 1987, the three communities entered into a Memorandum of Understanding For a Community Sponsored Automated Flood Warning System (City of Des Moines, 1987b) with the National Weather Service (NWS). This memorandum was undertaken for the purpose of defining a mutual assistance program designed to provide advance flood warning for the metropolitan area of Des Moines. The memorandum spelled out authorities, agreements, responsibilities, and funding of a system including the establishment of a maintenance fund.

During 1987 and 1988, the communities along with the assistance of the NWS evaluated the advance flood warning alternatives available. System formulation included Rock Island District involvement in the layout of the plan. The intent was to have an interim non-structural flood damage reduction measure the locals could use pending construction of the proposed Walnut Creek federal flood protection project by the Corps of Engineers. The result is a fully compatible,

approved flood warning system that has been incorporated into the needs of the current levee project. The cities of West Des Moines and Des Moines will receive credit for the system as part of the local cost-sharing agreement for the levee project.

In January 1989, the city of Des Moines released a Notice To Bidders and Specifications that detailed an early warning weather system based on the Automated Local Evaluation in Real Time (ALERT) system. The ALERT system was originally developed by the NWS California-Nevada River Forecast Center (RFC). The ALERT system consists of automatic reporting river and rainfall gages, a communications system based on line-of-sight radio transmission of data, a radio receiver, and a microprocessor network. The Des Moines ALERT system includes data analysis and display software to process, display, and control the quality of data. Additionally, the NWS RFC in St. Paul, Minnesota, developed hydrologic models and flood advisory tables that provide peak streamflow and time-to-peak forecasts based on antecedent moisture conditions and observed rainfall depths.

The Des Moines ALERT system is designed to provide real-time rainfall and stream level data for Walnut Creek on the western metro area and Four Mile Creek in the northeast portion of Des Moines. Additionally, Des Moines and Raccoon River streamflow data can be obtained. A layout map of the system is shown on Figure 3. The Walnut Creek portion of the ALERT system includes four remote rainfall gages and three stream gages. It is judged that this network, along with NWS weather information, provides adequate data for hydrologic evaluations. These gages report automatically to the data receiver and minicomputer communication center maintained at the NWS office at the Des Moines Airport.

This center can be interrogated at any time by the NWS or the locals to obtain needed data. As a system enhancement, each gage has a threshold parameter that sets off an alarm warning. The threshold for the rainfall gages is 2.0 inches in any 6-hour or less time period. A rate of rise exceeding one foot per one hour after reaching a pre-determined level will trigger the stream gage alarm warning. The local NWS office in conjunction with the St. Paul RFC judged these thresholds to be levels at which Walnut Creek flooding could be threatening. When these thresholds are met, the NWS will contact each city with an official notice of alert. From this point, the cities will access and monitor the ALERT system to keep abreast of developing conditions. Additionally, the NWS will continue to provide Flash Flood Warning bulletins and forecasted stages.

The ALERT system was installed during the summer of 1989 and became operational in November 1989. A March 13, 1990, rainfall and high-water event on Walnut Creek found the ALERT system to be fully operational and of great benefit to the metropolitan area. The city of Des Moines has agreed to be the designated lead agency and will coordinate the operation and maintenance of the system.

The RFC-developed Flood Advisory Table for the 63rd Street stream gage tied into the Walnut Creek portion of the ALERT system is attached as Figure 4. The Flood Advisory Table provides a rapid peak-stage forecast using antecedent moisture indexes produced by the Antecedent Precipitation Index (API) method. The forecasting relies on the Flood Advisory Table's precomputed flood stages for various antecedent conditions and average basin rainfall amounts. Figure 4 reflects project conditions, since levee construction impacts on the existing gage rating curve. This Flood Advisory Table technique has been provided to numerous NWS field offices and communities needing a method to produce an estimated flood peak (Pabst, 1986). The table also forecasts the timing of the rate of rise to flood stage and time of peak. The St. Paul RFC provides daily values of the Basin Index rainfall for its Des Moines Hydrologic Service Area. The locals can then access Walnut Creek rainfalls, enter the Flood Advisory Table with the Basin Index, and determine a flood peak estimate.

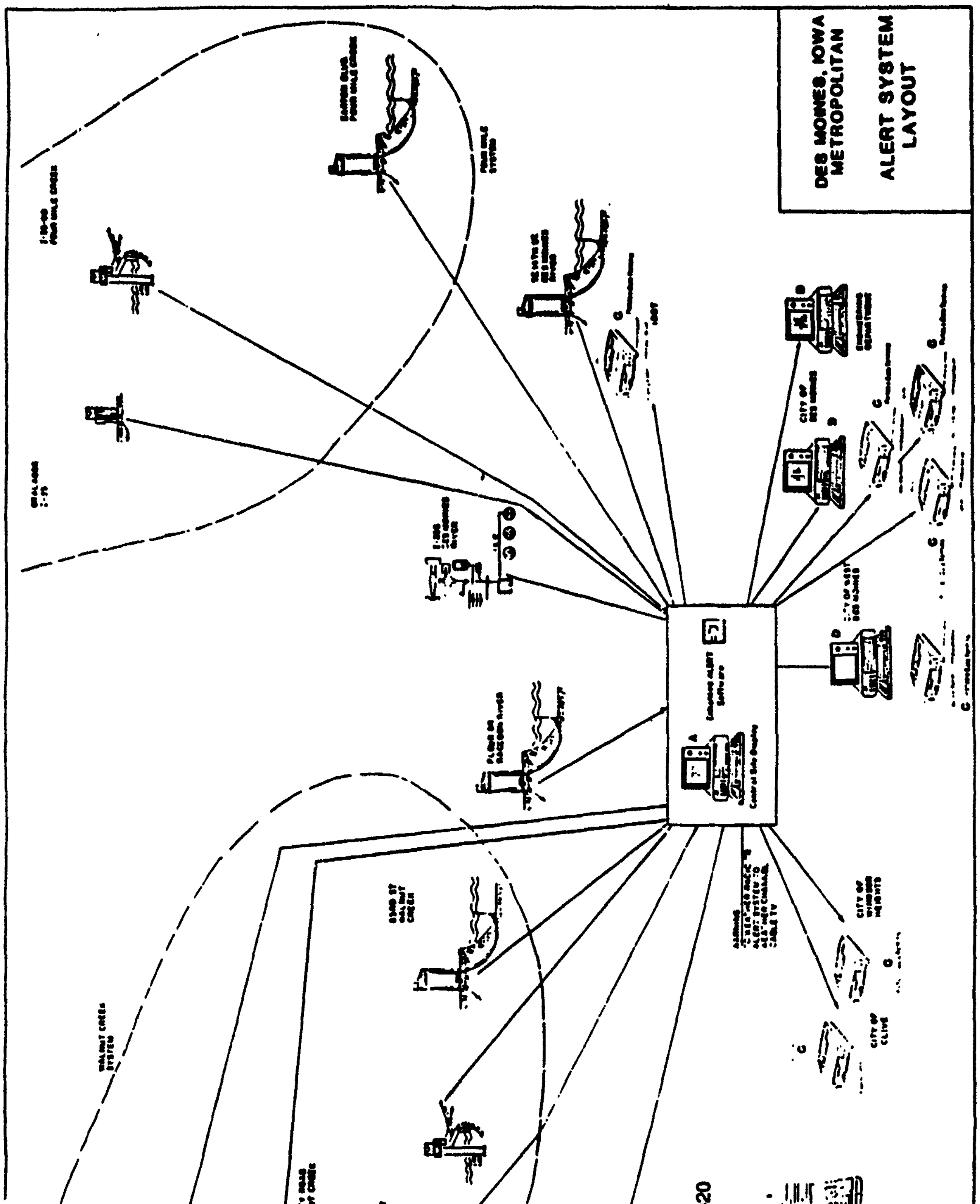


Figure 3

WALNUT CR.-W. DES MOINES,DES MOINES,IA   FLOOD ADVISORY TABLE

.......THIS TABLE IS DEVELOPED FOR THE LOCAL PROTECTION PROJECT...

FLOOD STAGE IS   13.0 FEET

LAG FROM BEGINNING OF THE TIME PERIOD OF HEAVIEST RAIN TO CREST IS 9 HOUR(S).

	STAGE FEET	DISCH. CFS	6 HOUR RAINFALL AMOUNTS (INCHES)									HRS FROM BGNG OF R/F TO FS
	10.0	1500	0.7	1.1	1.5	1.9	2.4	2.8	3.3	3.7	4.2	----
	11.0	1901	0.8	1.2	1.7	2.1	2.6	3.0	3.5	4.0	4.5	----
	12.0	2350	0.9	1.3	1.8	2.3	2.8	3.3	3.7	4.2	4.7	----
FS	13.0	2900	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	9
	14.0	3500	1.1	1.6	2.2	2.7	3.2	3.7	4.2	4.7	5.3	8
	15.0	4201	1.3	1.8	2.4	2.9	3.4	4.0	4.5	5.0	5.5	7
	16.0	5000	1.4	2.0	2.6	3.1	3.7	4.2	4.7	5.3	5.8	6
	17.0	5800	1.6	2.2	2.8	3.3	3.9	4.4	5.0	5.6	6.1	6
	18.0	7000	1.8	2.5	3.0	3.6	4.2	4.8	5.3	5.9	6.5	5
	19.0	9000	2.2	2.9	3.5	4.1	4.7	5.3	5.9	6.5	7.0	4
	20.0	12000	2.8	3.5	4.1	4.7	5.4	6.0	6.6	7.2	7.8	4
	21.0	15200	3.4	4.1	4.8	5.4	6.0	6.7	7.3	7.9	8.5	4
	22.0	19000	4.1	4.8	5.5	6.1	6.8	7.4	8.1	8.7	9.4	3
	23.0	23250	4.9	5.6	6.3	7.0	7.6	8.3	8.9	9.6	10.2	3
	24.0	27750	5.8	6.5	7.2	7.8	8.5	9.2	9.8	10.5	11.1	3
	25.0	32000	6.6	7.3	8.0	8.6	9.3	10.0	10.6	11.3	12.0	3

PEAK UNITGRAPH ORDINATE =   5420 CFS   FLOOD STAGE R.O. = 0.54 IN.

INSTRUCTIONS

THE AMOUNT OF RAINFALL REQUIRED TO PRODUCE FLOOD STAGE APPEARING IN THE RFC ADVISORY FIXES THE APPROPRIATE COLUMN OF RAINFALL--CREST STAGE VALUES TO BE USED DURING THE ENSUING PERIOD. ENTER THE TABLE AT THAT COLUMN AND FOLLOW UP OR DOWN TO THE AVERAGE OBSERVED RAIN TO OBTAIN THE CORRESPONDING CREST STAGE. USE ALL AVAILABLE RAINFALL REPORTS IN THE BASIN ABOVE THIS STATION AND AVERAGE.

EXAMPLE..THE RFC ISSUES AN ADVISORY THAT 2.0 INCHES IS THE AMOUNT OF RAINFALL THAT WILL PRODUCE A FLOOD STAGE AT THIS STATION. THAT NIGHT AN AVERAGE OF 3.0 INCHES OF RAIN FALLS OVER THE BASIN ABOVE THIS STATION. ENTER THE TABLE ABOVE WITH A VALUE OF 2.0 AT FLOOD STAGE,   13.0 FEET. FOLLOW DOWN THAT COLUMN TO 3.0 AND READ A STAGE OF   18.0 FEET. THE LAG TIME SHOWN AT THE HEAD OF THE TABLE IS 9 HOUR(S), SO THE PREDICTION IS A CREST OF   18.0 FEET, 9 HOUR(S) FROM THE BEGINNING OF THE PERIOD OF HEAVIEST RAINFALL. USING THE SAME COLUMN AND LINE, READ ACROSS TO THE RIGHTHAND COLUMN TO FIND THE   EXPECTED NUMBER OF HOURS FROM THE BEGINNING OF RAINFALL TO THE TIME THAT THE STREAM REACHES FLOOD STAGE. HENCE, IF THE HEAVY RAIN STARTED AT 7PM THE STREAM WOULD REACH FLOOD STAGE AT 12PM AND CREST AT 4AM THE FOLLOWING MORNING.

MAXIMUM STAGE OF RECORD

WALNUT CR.-W. DES MOINES,DES MOINES,IA   NCRFC   MAY 1990   DES HSA

Figure 4

The above mentioned API method is a running daily Basin Index that reflects the current moisture level of the watershed. The API is increased by precipitation and decreased by a seasonally dependent factor. The API for a given day is then used in the basin's rainfall-runoff relationships along with the time of year, the rainfall depth, and the storm duration. The resulting storm runoff is then applied to the appropriate unit hydrograph watershed model. The watershed modelling done by the RFC has been coordinated with the Rock Island District, and additional HEC-1 modelling has verified the RFC methods.

The NWS, as part of the development of the Flood Advisory Tables, determined bankfull, alarm, and flood stages for the three stream gages on Walnut Creek. The two upstream gages are new locations, whereas the downstream 63rd Street gage has been in existence since October 1971. The various stages along with other pertinent gage information are listed in Table 1.

Table 1

Stream Gage Data For The Des Moines ALERT System

Location	Sensor Number	Gage Datum (NGVD)	Bankfull Stage	Flood Stage	Alarm Stage
Waukee 3NE	1013	897.00	7	9	7
I-35/80	1003	850.00	7	11	9
63rd Street	1051	801.04	11	13	-

The flood stage of 13 feet at the 63rd Street gage is consistent with what has been used in the past. The locals are familiar with this 13-foot stage and use it as a trigger point in flood-fighting actions. It will continue as the trigger forecasted stage at which the initial operations of the proposed flood protection project will need to be initiated.

During a 100-year design storm, the ALERT system will give the locals 2-3 hours of advance warning time to make storm sewer gatewell closures at the 13-foot flood stage. Additionally, each storm outlet will be equipped with a flapgate. The first roadway/railroad closure will have an advance warning time of four hours. Each successively higher closure will have increasingly longer advance warning times. The warning times are based on actual reported rainfall depths and predetermined Walnut Creek watershed rainfall/runoff relationships. Thus, the warnings should be a reliable indication of an impending flood. False warnings will be minimized based on this system. Existing severe weather forecasting will place the locals' emergency services personnel on alert three hours or more prior to the above warning times.

Flood Response. The Des Moines ALERT system provides the critical advance flood warning. However, this information is of little to no benefit without a flood response plan. The Rock Island District and the cities of West Des Moines and Des Moines fully realize this fact. The cities are updating their flood response plans to include the addition of the ALERT system. Both cities have adopted flood response plans that detail objectives, responsibilities, emergency operation procedures, specific flood-fighting measures, and closure elevations for existing facilities. The Rock Island District will be assisting the cities in updating of their respective plans to include the addition of the proposed local flood protection project. Each city's flood response plans are summarized below.

The city of Des Moines has a Flood Emergency Plan (City of Des Moines, 1987a) published by its Public Works Department. The plan outlines an operating procedure and the duties and

responsibilities of the Public Works and Engineering Departments when responding to a flood emergency. The Chief Administrator of all emergency operations is the City Manager or his designated representative. The Director of Operations is the Public Works Director or his designee, and he is directly respor :ible to the City Manager for coordinating the entire emergency program with all other city departments and outside agencies.

The plan establishes a 24-hour Control Center, where Des Moines has its ALERT system base station computer. It also lists specific duties of divisions of the Public Works and Engineering Departments, such as the Sewer Maintenance Division is to close flood gates on needed levee closures. The Flood Emergency Plan also includes a listing of key city staff along with home and work telephone numbers. The plan also contains a location-by-location description, action, closure elevation, and other pertinent information on every storm sewer, sanitary sewer, levee closure, pump station, and street closure needed within the city that may be affected by flooding.

The city of West Des Moines has an adopted Flood Hazard Operational Plan (City of West Des Moines, 1979) developed by its Department of Civil Defense. The plan provides a list of guidelines for the organization, procedures, and resources necessary for public safety in the event of threatened or actual flooding. An Emergency Operations Staff is created for the purpose of working as a management team in generally controlling and supervising overall flood emergency actions. The Mayor, City Manager, Civil Defense Director, Police Chief, City Engineer, Public Works Superintendent, Waterworks General Manager, and Fire Chief compose the staff of this team. Basic responsibilities of the Emergency Operations Staff include sizing up the situation based upon reports from the field, th ALERT system, and other sources, determining the strategy and tactics that will be used aling with the flood emergency, and exercising direction and control over local forces. The We s Moines plan also lists specific departmental responsibilities.

Both cities' flood response plans will be updated to include the specific information needed to operate the Raccoon River and Walnut Creek flood protection project. Departmental responsibilities will remain largely the same. The updating will include a utilization plan for the ALERT system. The flash flood nature of Walnut Creek will require on-duty city personnel to implement the necessary closure operations of the project. This will include personnel from the police, fire, engineering, and public works departments responding to the real-time data provided by the ALERT system. Alert and mobilization stages will be established once the NWS isssues a Flash Flood Watch bulletin for the Walnut Creek basin or when an alarm warning parameter is exceeded in the ALERT system. The NWS bulletin is typically issued when meteorological conditions exist that could result in severe rainfall. Thus, appropriate personnel will have been placed on alert prior to the start of any severe storm and potential flood situation. The city of Des Moines has also mentioned monitoring NWS Quantitative Precipitation Forecast (QPF) maps to provide advance information on a possible threatening thunderstorm.

The March 13, 1990, Flash Flood Watch bulletin issued by the NWS found the ALERT system working as designed and the cities' flood response plans to be operational. Adequate personnel within each city's staff were cognizant of their required duties and adequately trained in interrogating the ALERT system.

Study Results

Summary of Gate Closure Operations. The operation of the overall flood protection project can be subdivided several ways; Raccoon River and Walnut Creek, exterior and interior closures, and Des Moines and West Des Moines. This section will discuss primarily the exterior flooding levee gate closures and the interior drainage facility closures on the Walnut Creek portion of the project.

Flood warning and response on the Raccoon River have been institutionalized by both cities via past flood-fighting measures.

The formulation and design of the West Des Moines - Des Moines local flood protection project has resulted in a plan that includes three interior drainage and five roadway/railroad levee closures on the Walnut Creek portion of the project. The cities will be placed on alert whenever the NWS issues a Flash Flood Warning bulletin or when an alarm warning parameter in the ALERT system in exceeded. The locals will then monitor the ALERT system and NWS forecasts on a regular basis to keep abreast of any developing flood situation. Additionally, initial mobilization of emergency responses will be coordinated in this alert phase.

Upon observation through the ALERT system that Walnut Creek basin rainfalls are exceeding the NWS Flood Advisory Table basin index rainfall to cause flood stage or flood stage is reached at any of the three stream gages, official mobilization will commence. Interior drainage facilities will need initial flood response.

Presently, there are twelve interior outlet locations draining to Walnut Creek. These twelve outlets are recommended to be collected via interceptor storm sewers and surface grading into three outlets. The interior drainage outlets will be equipped with gatewell structures at the levee. Each gatewell will have a sluice gate for positive closure during periods when flood levels exceed interior ponding levels. The sluice gates can be readily closed in fifteen to twenty minutes each.

The 63rd Street gage on Walnut Creek will serve as the focal point for when closure of each gatewell should be made. The gatewells are located at Station 147+00 by Valley Drive, at Station 187+85 on 63rd Street, and at Station 202+70 on Hoak Drive. The city of Des Moines will be responsible for the Valley Drive gatewell with the other two gatewells being West Des Moines' responsibility. A stage of 13.5 feet will be the closure decision point. Since gatewell closures typically occur during periods of rainfall, actual closure should occur when interior outflow ceases and floodwater backup commences at each location. The Walnut Creek gatewell closure data are tabulated on Table 2. The Raccoon River gatewell closures are also listed on Table 2.

The first two roadway/railroad closures will be the Norfolk Southern Railroad swing gate by 63rd Street and the Wheeler Lumber service road swing gate. The city of Des Moines will have primary responsibility for the railroad closure, and West Des Moines will have the lead on the Wheeler Lumber closure. Both of these closures have sill elevations equivalent to a flood stage of 18.0 feet. These closures can be made in one hour each with the advance warning time of four hours available.

Subsequent closure sill elevations on the Grand Avenue slide gate and the 63rd Street swing gate are overtopped when a flood stage of 19.5 feet is exceeded. Des Moines will be responsible for the Grand Avenue closure and West Des Moines will handle the 63rd Street closure. These closures can be made in 1.5 hours each with an advance warning time of five hours available.

The final railroad swing gate closure has a sill elevation equivalent to a flood stage of 20.0 feet. West Des Moines will be responsible for this closure. This closure can be made in one hour with an advance warning time of five hours available.

The decision to make closure at each of the above levee locations should be made whenever the Walnut Creek stage is forecasted to be within two feet of the sill elevation of each closure. This will add a safety factor to all closure decision points in the event actual stages are higher than the forecasted stage. Thus, the initial roadway/railroad closures will be made at a forecasted stage of

TABLE 2

SUMMARY OF GATEWELL CLOSURES
WEST DES MOINES-DES MOINES, IOWA

STATION	LOCATION	AFFECTED BY	CITY	GATEWELL DIM. (DIA,IN) (HxW,FT)	TYPE	ALERT STAGE GAGE (FT) #1	ALERT STAGE CITY (FT) #2	ALERT Q (CFS)	ALERT FREQ (YR)	MOBILIZATION STAGE GAGE (FT) #1	MOBILIZATION STAGE CITY (FT) #2	MOBILIZATION Q (CFS)	MOBILIZATION FREQ (YR)	ZERO DAMAGE LEVEL STAGE GAGE (FT) #1	ZERO DAMAGE LEVEL STAGE CITY (FT) #3	ZERO DAMAGE LEVEL Q (CFS)	ZERO DAMAGE LEVEL FREQ (YR)	CLOS. TIME (HR)	AVAIL. TIME (HR)	SAFETY FACTOR
13+50	LINCOLN RD	RACCOON	WDM	48"	SLUICE	13	31	8500	1.25	22	38.5	28,000	9	26.7	44.7	60,000	200	.25	15+	60+
43+60	FAIR MEADOWS	RACCOON	WDM	2-72"	SLUICE	13	31	8500	1.25	20	37	23,000	5	26.2	44.2	45,000	50	.50	15+	30+
59+50	8th ST.	RACCOON	WDM	6'x6'	SLUICE	13	31	8500	1.25	13	31	8,500	1.25	17.2	35.2	17,000	3	.25	15+	60+
67+50	6th ST	RACCOON	WDM	48"	SLUICE	13	31	8500	1.25	13	31	8,500	1.25	17.2	35.2	17,000	3	.25	15+	60+
81+50	63rd ST	RACCOON	WDM	5'x7'	SLUICE	13	31	8500	1.25	13	31	8,500	1.25	17.2	35.2	17,000	3	.50	15+	30+
100+45	63rd ST	RACCOON	DM	24"	SLUICE	13	31	8500	1.25	16	34	14,000	2	17.2	35.2	17,000	3	.25	15+	60+
103+50	C-NW RR	RACCOON	DM	36"	SLUICE	13	31	8500	1.25	13	31	8,500	1.25	17.2	35.2	17,000	3	.25	15+	60+
147+00	VALLEY DR	WALNUT	DM	48"	SLUICE	Nat. Weather Serv.				13.5	40.7	Forecasted 3,000	2	19.0	35.2	9,500	20	.25	2	8
187+85	63rd ST	WALNUT	WDM	5'x7'	SLUICE	Flood Warning Bulletin or				13.5	40.7	Forecasted 3,000	2	14.5	40.2	3,800	3	.33	2	6
202+70	HOAK DR	WALNUT	WDM	24"	SLUICE	ALERT System Alarm Warning				13.5	40.7	Forecasted 3,000	2	15.5	43.2	4,400	4	.25	2	8

#1 Stage @ Van Meter Gage, 0.0 = 841.2 NGVD
Stage @ 63rd Street Gage, 0.0 = 801.0 NGVD
#2 Stage @ 63rd Street, Racoon/Walnut, 0.0 = 773.8 NGVD
#3 Stage @ Site Location, 0.0 = 773.8 NGVD

Table 2

16.0 feet. This information is tabulated on Table 3 along with the Raccoon River closure information. Since October 1971, a stage of 16.0 feet has been exceeded on four occasions.

Conclusions

Historically, the cities of West Des Moines and Des Moines have had to deal with Raccoon River and Walnut Creek flooding. Some low-level flood protection facilities exist on the Raccoon River in the project area, and other comprehensive systems are in place downstream in Des Moines. Thus, flood warning and response concerning Raccoon River flooding is already functional due to existing institutions. The operation procedures for the Raccoon River portion of the project will be readily incorporated into the existing flood emergency plans resulting in a safe and functional project for the locals to operate.

The flood warning and response on the Walnut Creek portion is more critical due to the flash flood nature of the stream. However, the Rock Island District and the local sponsors have foreseen this and have installed an early weather warning system centered around the ALERT system and existing NWS severe weather forecasting. The formulation and design the flood protection project's levee gate closures and their subsequent operation are compatible with the advance warning provided by the flood warning system.

The end result is an ideal scenario. The locals have an operational flood warning system that they are intimately familiar with. This situation has occurred 2-4 year before the flood protection project is constructed and becomes operational. Most of the warning system's startup problems will be resolved and institutional responses to the warning established in an adopted flood emergency plan. The formulation and design of the levee gate closures is consistent with the prevailing advance warning times available on each stream, and the operation of the overall project is within the resources of each city.

References

Flood Emergency Plan, Department of Public Works, City of Des Moines, Iowa, 1987

Flood Hazard Operational Plan, Department of Civil Defense, City of West Des Moines, Iowa, 1979

Memorandum of Understanding For Community Sponsored Automated Flood Warning System, City of Des Moines, Iowa, 1987

State-of-the-Art Flood Forecasting Technology, Art Pabst, HEC, U.S. Army Corps of Engineers, 1986

TABLE 3

SUMMARY OF ROAD AND RAILROAD CLOSURE
WEST DES MOINES-DES MOINES, IOWA

STATION	LOCATION	AFFECTED BY	CITY	CLOSURE DIM. (HxW,FT)	TYPE	ALERT STAGE GAGE (FT) #1	ALERT STAGE CITY (FT) #2	ALERT Q (CFS)	ALERT FREQ (YR)	MOBILIZATION STAGE GAGE (FT) #1	MOBILIZATION STAGE CITY (FT) #2	MOBILIZATION Q (CFS)	MOBILIZATION FREQ (YR)	SILL OVERTOPPING STAGE GAGE (FT) #1	SILL OVERTOPPING STAGE CITY (FT) #3	SILL OVERTOPPING Q (CFS)	SILL OVERTOPPING FREQ (YR)	CLOS. TIME (HR)	AVAIL. TIME (HR)	SAFETY FACTOR
9+65	C-RR&P RR	Raccoon	WDM	1.0x40	Sandbag	13	31	8500	1.25	27.2	43.2	60,000	200	29.2	47.2	80,000	500+	2	15+	7.5+
39+33	Lincoln Rd	Raccoon	WDM	7.0x35	Swing Gate	13	31	8500	1.25	20.0	37.0	23,000	5	22.0	40.0	32,000	15	1.5	15+	10+
89+76	63rd St.	Raccoon	WDM	1.7x50	Sandbag	13	31	8500	1.25	23.5	41.5	45,000	50	25.5	43.5	60,000	200	3	15+	5+
103+57	C & NW RR	Raccoon	DM	4.3x100	Swing Gate	13	31	8500	1.25	20.5	38.5	28,000	9	21.9	39.9	40,000	33	1.5	15+	10+
											Forecasted									
184+61	Grand Ave.	Walnut	DM	6.3x65	Slide Gate	Nat. Weather Serv.				17.5	44.7	6,600	8	19.5	44.3	10,000	20	1.5	5	3.3
											Forecasted									
189+46	N-S RR	Walnut	DM	7.7x15	Swing Gate	Flood Warning				16	43.2	5,000	4	18	44.2	7,700	10	1	4	4
											Forecasted									
192+71	63rd St	Walnut	WDM	5.9x70	Swing Gate	Bulletin or				17.5	44.7	6,600	8	19.5	46.7	10,000	20	1.5	5	3.3
											Forecasted									
218+00	Wheeler	Walnut	WDM	7.3x20	Swing Gate	ALERT System				16	43.2	5,000	4	18	47.2	7,300	10	1	4	4
											Forecasted									
219+67	N-S RR	Walnut	WDM	5.5x20	Swing Gate	Alarm Warning				18	45.2	7,300	10	20	49.0	11,500	25	1.5	5	3.3

#1 Stage @ Van Meter Gage, 0.0 = 841.2 NGVD
Stage @ 63rd Street Gage, 0.0 = 801.0 NGVD
#2 Stage @ 63rd Street, Racoon/Walnut, 0.0 = 773.8 NGVD
#3 Stage @ Site Location, 0.0 = 773.8 NGVD

Table 3

SAFETY CONCERNS FOR LEVEES AND RINGWALLS

by

Larry E. Holland[1]

Introduction

Study Purpose. Levee and floodwall local protection projects do not generally provide an ultimate level of protection and can be overtopped by flood events exceeding the recommended level of protection. In addition, such projects generally require the local sponsor of the flood protection project to coordinate and implement timely operation of all closure structures to ensure the overall flood protection project functions as intended. The purpose of this paper is to discuss the functional performance and safety related aspects of a levee/floodwall and ringwall system for the city of Buena Vista, Virginia. A feasibility level report completed in 1990 recommended construction of a levee/floodwall and ringwall system for the city of Buena Vista. This plan would provide a level of protection slightly greater than the 1% flood event and was determined to be the National Economic Development Plan (NED).

Key Issues. The major issues concerning the levee and ringwall system for Buena Vista centered on the ability of the locals to operate the numerous closures in a timely manner and the ability of the locals to implement an evacuation plan should overtopping of the line of protection became imminent. These concerns for the recommended local protection project were considered critical to the plan selection, given the relatively small size of the community, the complexity of the local protection project, and the relatively fast rising nature of the Maury River. These concerns were expressed by headquarters personnel at a technical review conference held prior to the Feasibility Review Conference (FRC).

Summary of Findings. Analysis of available data and significant coordination with the locals demonstrated that, although advance warning times are short and there is a potential for a significant number of false alarms, the locals can accomplish the closure operations in a timely manner for the recommended plan and, if required, can complete evacuation of the protected area should overtopping of the line of protection become imminent.

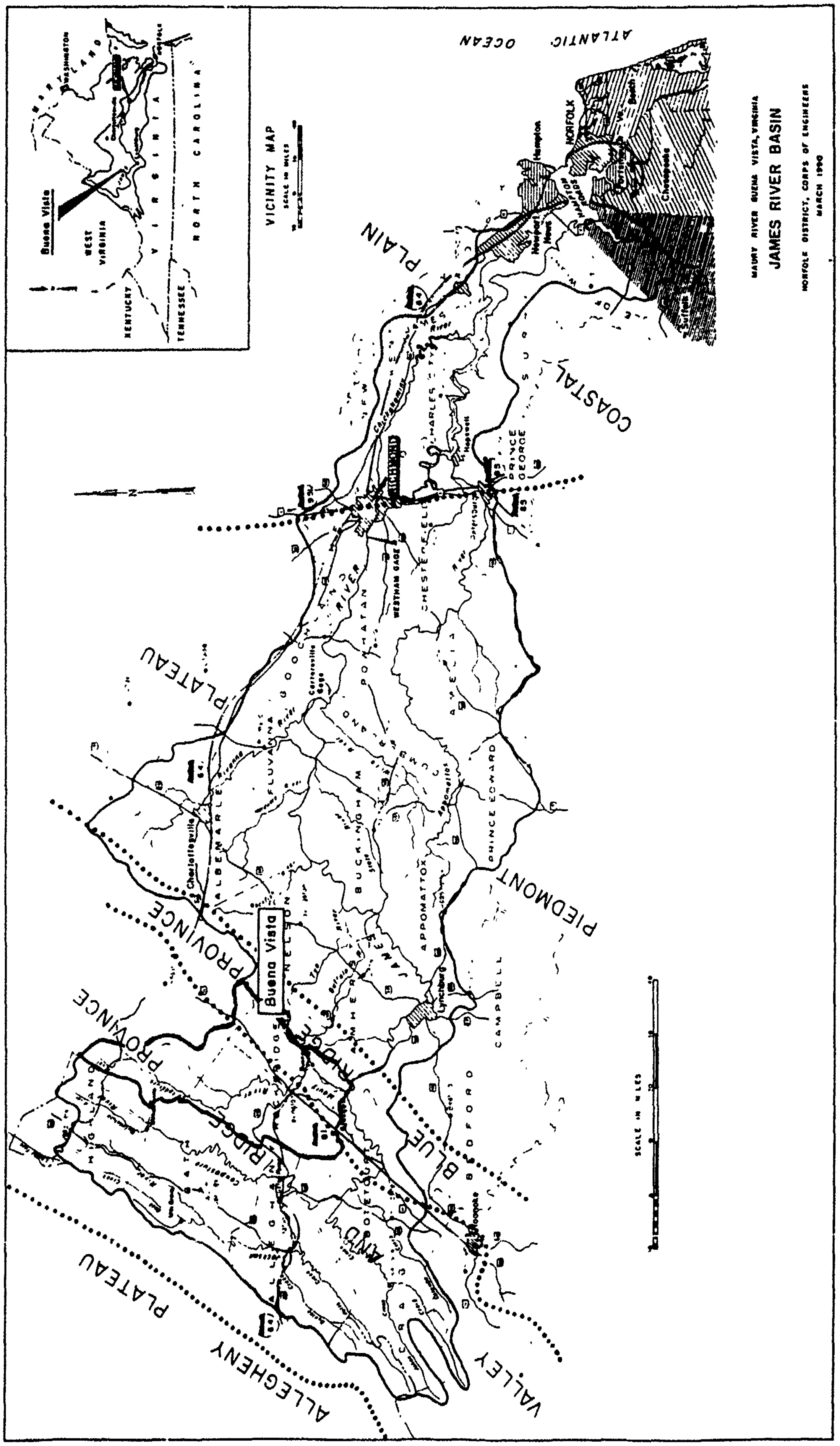
Physical Setting and Available Data

Basin Description. The Maury River Basin is located in the west central portion of the Commonwealth of Virginia and lies within two physiographic regions known as the Valley and Ridge Province and the Blue Ridge Province, as shown on Figure 1. The watershed drains a total area of 840 square miles, of which slightly more than 650 square miles lies upstream of the city of Buena Vista.

The Allegheny Mountains form the western boundary of the basin while the Blue Ridge Mountains form the eastern boundary. Topography in the basin is generally rolling, becoming steep on the mountains. Elevations generally range from 1,000 to 1,500 feet in the valley up to 4,000 feet on the mountains. The city of Buena Vista lies on the edge of the valley floor at the base of the western slopes of the Blue Ridge Mountains.

About two-thirds of the Maury River Basin is wooded and includes portions of the Jefferson and George Washington National Forests. The remainder of the basin is devoted to agricultural uses, including pastures used for the grazing of livestock and cropland, and the three relatively small urban developments of Lexington, Buena Vista, and Glasgow.

[1]Chief, Hydraulics and Hydrology Branch, Norfolk District, U.S. Army Corps of Engineers



MAURY RIVER BUENA VISTA, VIRGINIA
JAMES RIVER BASIN
NORFOLK DISTRICT, CORPS OF ENGINEERS
MARCH 1990
VICINITY MAP
Buena Vista
ATLANTIC OCEAN
COASTAL PLAIN
PIEDMONT PLATEAU
PIEDMONT PROVINCE
BLUE RIDGE PROVINCE
VALLEY AND RIDGE PROVINCE
ALLEGHENY PLATEAU
NORFOLK
Hampton
Newport News
Chesapeake
ALBEMARLE
FLUVANNA
BUCKINGHAM
APPOMATTOX
CAMPBELL
BEDFORD
NELSON
PRINCE EDWARD
POWHATAN
CUMBERLAND
AMELIA
PRINCE GEORGE
CHESTERFIELD
WESTHAM GAGE
JAMES RIVER
Charlottesville
Lynchburg
SCALE IN MILES

FIGURE 1

Project Area. The city of Buena Vista is located in the western part of the State of Virginia on the Maury River approximately 11 miles above the confluence with the James River, as shown on Figure 1. The city lies at the foot of the steep western slopes of the Blue Ridge Mountains and encompasses an area of approximately 3 square miles. In 1980, the city had a population of approximately 6,700 persons and has remained relatively constant over the last 10 years. A substantial portion of the city's railroad, industrial, public, and commercial properties lie within the Maury River flood plain. Five local streams, with drainage areas ranging from slightly more than 1 square mile to nearly 6 square miles drain through the city of Buena Vista and empty into the Maury River within the project limits, as shown on Figure 2.

Project Description. The recommended plan of flood control for the city of Buena Vista as shown on Figure 2 includes a combination levee/floodwall for the main portion of the city including associated interior flood control facilities. A separate ringwall segment was also recommended for separate industrial facilities located near the upstream end of the city. The separate ringwall was included in the recommended plan rather than a continuous line of protection to avoid crossing Chalk Mine Run and Long Hollow Run, which have a total drainage area of approximately 7.2 square miles. The selected plan would provide protection for a major portion of the city from floods up to the August 1969 flood of record (90,000 c.f.s.) which has approximately a 0.87% chance of occurring in any given year.

Climate and Storm Characteristics. The climate of the Maury River Basin can be described as temperate with relatively mild winters and warm summers. The average annual temperature in the basin is approximately 56 degrees with extremes below zero and above 100 degrees experienced on rare occasions.

Precipitation is well distributed throughout the year with the maximum occurring in the spring and summer months and the minimum in the fall and winter months. Average annual precipitation over the basin averages approximately 38 inches, which is approximately 4 inches below the average precipitation for the entire James River Basin. Average annual snowfall for the Maury River Basin is approximately 22 inches, but accumulation is not generally considered a flood-producing factor.

The Maury River Basin is subject to flood-producing storms throughout the year, but the frequency of flooding is generally greater during the winter and spring months. The sustained winter and spring storms generally produce the large floods, particularly along the main stem of the river. Hurricanes and other tropical disturbances occasionally move far enough inland to affect the Maury River Basin and surrounding areas. Although they have generally lost their identity as hurricanes by this time, the remaining low pressure center in conjunction with the lifting effects of the steep mountain ranges can produce unusually heavy rainfall within the Maury River Basin. The two largest floods on record at Buena Vista were produced by the remnants of tropical hurricanes. Intense summer thunderstorms, which are generally more local in nature, can produce flood conditions within the basin, particularly along the tributary streams.

Streamflow Data. Streamflow records for the Maury River Basin are collected and maintained by the U.S. Geological Survey in cooperation with the Virginia State Water Control Board. The locations of these gaging stations are shown on Figure 3. The Maury River near Buena Vista gage is the nearest stream gage to the project area and is located 2.8 miles northwest of Buena Vista and 0.5 miles downstream of the South River. The gage responds to a drainage area of 646 square miles (compared to a drainage area of approximately 650 square miles at the upstream project limits) with continuous records dating to 1938. The Maury River Basin, due to the basin characteristics, is somewhat typical of most streams in the region with relatively high rates of rise (can reach flood stage in a matter of hours) and relatively short durations of flooding (generally above flood stage for less than 1 to 1-1/2 days). Table 1 provides a summary of the history of flooding on the Maury River in the vicinity of Buena Vista.

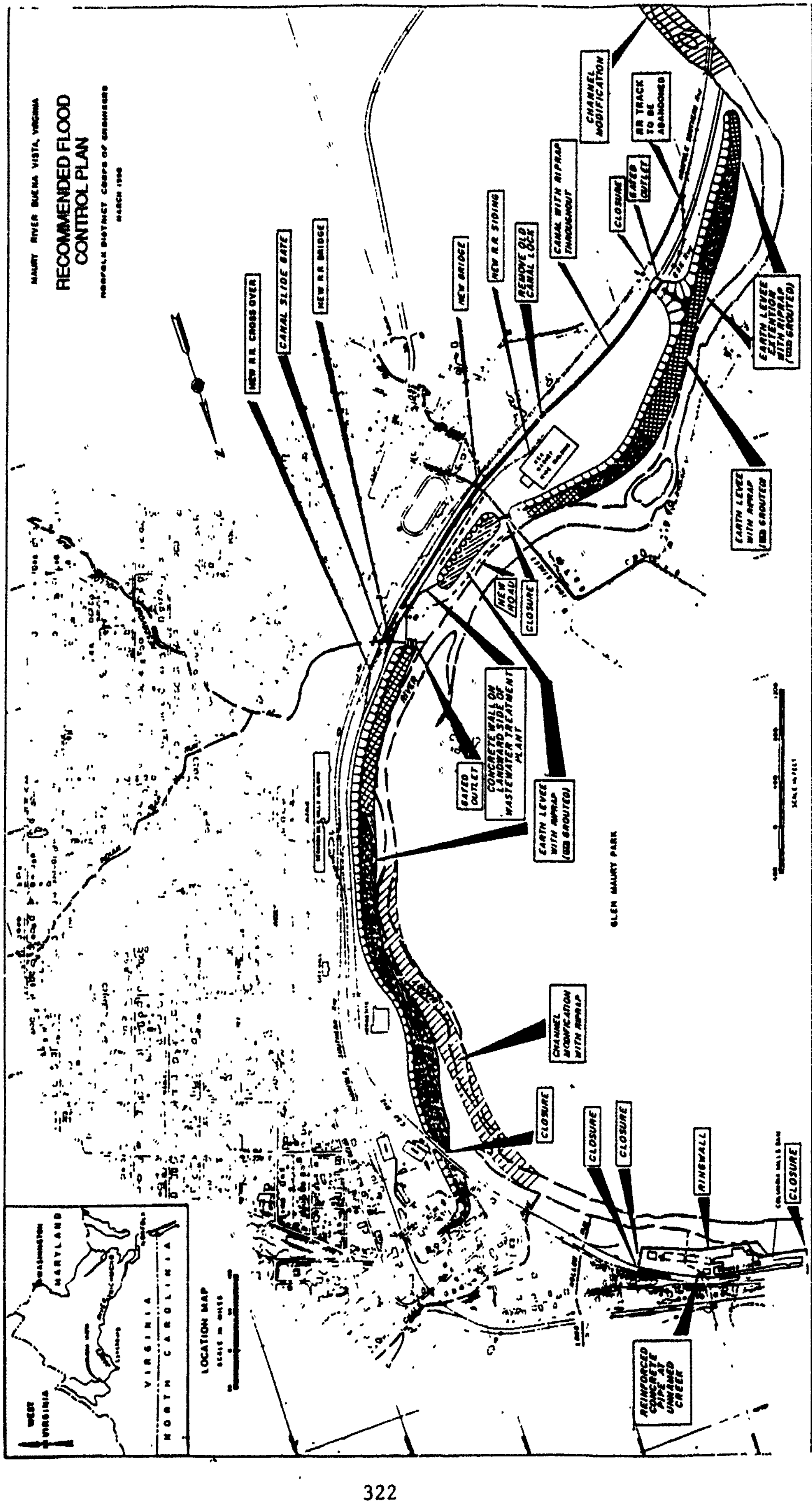

MAURY RIVER BUENA VISTA, VIRGINIA
RECOMMENDED FLOOD CONTROL PLAN
NORFOLK DISTRICT CORPS OF ENGINEERS
NEW R.R. CROSS OVER
CANAL SLIDE GATE
NEW R.R. BRIDGE
NEW BRIDGE
NEW R.R. SIDING
REMOVE OLD CANAL LOCK
CANAL WITH RIPRAP THROUGHOUT
CLOSURE
GATED OUTLET
RR TRACK TO BE ABANDONED
CHANNEL MODIFICATION
EARTH LEVEE EXTENTION WITH RIPRAP
EARTH LEVEE WITH RIPRAP
NEW ROAD
CLOSURE
CONCRETE WALL ON LANDWARD SIDE OF WASTEWATER TREATMENT PLANT
GATED OUTLET
EARTH LEVEE WITH RIPRAP
GLEN MAURY PARK
CHANNEL MODIFICATION WITH RIPRAP
CLOSURE
CLOSURE
CLOSURE
RINGWALL
CLOSURE
REINFORCED CONCRETE PIPE AT UNNAMED CREEK
LOCATION MAP
WEST VIRGINIA
MARYLAND
WASHINGTON
VIRGINIA
NORTH CAROLINA

FIGURE 2

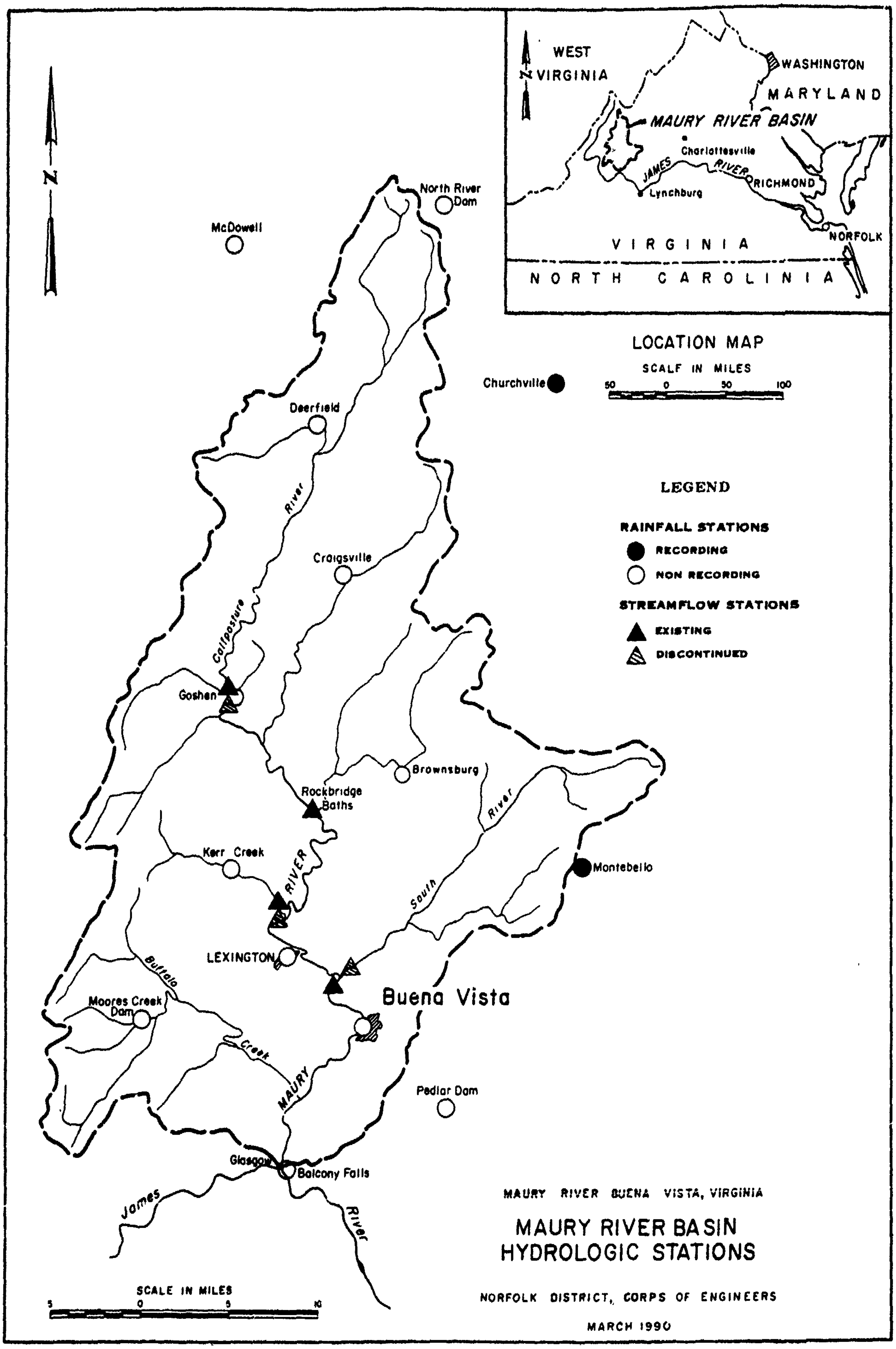

WEST
VIRGINIA
WASHINGTON
MARYLAND
MAURY RIVER BASIN
Charlottesville
JAMES
RIVER
RICHMOND
Lynchburg
NORFOLK
VIRGINIA
NORTH CAROLINIA
LOCATION MAP
SCALF IN MILES
50 0 50 100
North River Dam
McDowell
Churchville
Deerfield
Craigsville
Calfpasture
River
Goshen
Brownsburg
Rockbridge Baths
Kerr Creek
RIVER
South
River
Montebello
LEXINGTON
Buena Vista
Moores Creek Dam
Buffalo
Creek
MAURY
Pedlar Dam
Glasgow
Balcony Falls
James
River
SCALE IN MILES
5 0 5 10
LEGEND
RAINFALL STATIONS
RECORDING
NON RECORDING
STREAMFLOW STATIONS
EXISTING
DISCONTINUED
MAURY RIVER BUENA VISTA, VIRGINIA
MAURY RIVER BASIN
HYDROLOGIC STATIONS
NORFOLK DISTRICT, CORPS OF ENGINEERS
MARCH 1990

FIGURE 3

Table 1. FLOOD HISTORY, MAURY RIVER NEAR BUENA VISTA, VIRGINIA

Flood	Gage Height, ft (a)	Discharge, c.f.s.	Flood Event, %
August 1969	31.23	90,000 (b)	0.87
November 1985	26.30	72,100	1.4
March 1936	22.00	50,000	3.3
June 1972	17.10	27,800	12.1
September 1950	16.20	22,400	18.5
October 1961	16.20	22,400	18.5
January 1978	15.46	22,300	19.0
May 1942	16.00	21,700	20.0
December 1973	14.63	19,700	23.5
June 1982	14.55	19,400	24.0

(a) Flood Stage=17.0 feet. Gage zero=846.58 feet, NGVD
(b) Estimates by the USGS place the peak discharge at 105,000 c.f.s.; however, indirect methods and backwater studies by the Norfolk District, U.S. Army Corps of Engineers indicate a discharge of 90,000 c.f.s. is more appropriate for the reach of the Maury River through the city of Buena Vista.

Study Approach and Results

Operation of the project for flood control involves the timely operation of all closure structures, including the gravity outlets and the street and railroad closures. The city of Buena Vista will be responsible for coordinating and implementing these closure operations. In addition, the city will be responsible for implementing an evacuation plan should overtopping of the line of protection become imminent. By formal correspondence, the City Manager indicated the city's awareness of their responsibilities concerning operation of the project and that the city is capable of accomplishing the necessary closures and operations during flood periods. The following discussions summarize the information provided by the city of Buena Vista concerning their flood emergency preparedness, identify the actions necessary to accomplish the closure operations and evacuations, and describe an analysis of available warning times.

Flood Emergency Preparedness. The city of Buena Vista has experienced major flooding in 1969 and 1985. As a result of this relatively recent flooding, the community has a heightened level of public awareness of the existing flood threat and the city is especially cognizant of forecasted and actual heavy rainfall in the area and of any rises in the Maury River. This is borne out by the city's response in advance of the passage of Hurricane Hugo in September 1989. The city's Emergency Operations Plan (EOP) was effected and city forces were mobilized as several preparatory actions were accomplished. Fortunately, the community was spared another major flood event, but these actions illustrate the city's and the community's willingness and desires to respond to perceived flood threats.

The Police Department dispatcher's office presently serves as the city's Emergency Operation's Center (EOC) and is responsible for monitoring river levels of the Maury River. This office is staffed 24 hours a day and has a standing procedure for monitoring the Maury River levels and issuing appropriate notifications of possible flooding within Buena Vista. If conditions warrant, additional off duty dispatchers and personnel are called in to operate the EOC and assist with the river level monitoring and notification procedures. River levels for the Maury River near Buena Vista gage, located several miles upstream from the city, are presently determined telephonically.

The city of Buena Vista is also presently participating in a flood warning system developed by the National Weather Service. The Integrated Flood Observing and Warning System (IFLOWS) relies heavily on an extensive network of rainfall data-gathering sites and limited stream gages to provide real-time

warnings of the potential for flooding for localities with relatively short flood warning times. Although the IFLOW system for Buena Vista is not fully operational at this time due to a communication problem which prevents the city from receiving transmissions directly from the gages, the system should be fully operational by the time construction of a flood control project at Buena Vista would be completed. At this time, the state EOC in Richmond is receiving the rain gage information and can provide information to the city of Buena Vista if needed. Once fully operational, the network will provide the city of Buena Vista with advance warning time in addition to that discussed in this paper.

The City Manager is presently notified directly by the Police Department dispatcher's office when conditions warrant and he in turn notifies his appropriate department heads, including the Public Works Department. This notification procedure is considered sufficient for project operations once a flood control project is constructed at Buena Vista. The city's Public Works Department will be assigned the task of accomplishing the necessary closure operations once the flood control project is completed. The Public Works Department consists of 27 paid personnel with the majority of personnel presently living in or close proximity to Buena Vista. Therefore, these personnel are quickly available during an emergency and could be expected to respond in less than 1/2 hour from the time a standby alert is issued. As discussed earlier, additional advance warning time can be issued based on rainfall information provided by IFLOWS.

Closure Operations. The recommended plan includes ten separate closure structures. Four of these closures are located in the ringwall portion at the northern end of the project. The city has indicated that Georgia Bonded Fibers (GBF) will be asked to assume the operational responsibility of implementing these four closures. GBF has the manpower, equipment, and incentive necessary to accomplish the four closures during flood warning periods. GBF's responsiveness to a flood threat is indicated by the fact that they have, when provided sufficient advanced warning of an impending flood threat, removed equipment from the threatened area. GBF will receive flood alert notifications directly from the Police Department dispatcher's office at the same time the City Manager is notified.

Of the remaining six closures, two are flap gates which only require visual inspection to ensure they are operating properly and not blocked by debris. The teams assigned to inspect and operate these gates can also be utilized to accomplish other closures at those sites. For instance, after inspecting the flap gates on the outlet structure at Indian Gap Run, the same team can open the interior flood control canal diversion gate at that site. Also, the team that inspects the flap gates on the outlet structure at the downstream end of the diversion canal can, at the appropriate time, make the railroad closure at the downstream end of the project. Only two additional teams, for a total of four teams in addition to the GBF personnel, are necessary to accomplish the railroad closure near Hermitite and the 10th Street Bridge closure. The city has indicated that three or more personnel from the Public Works Department will be assigned to each crew to ensure that the necessary number of personnel are available and will respond to any call out.

The closure structures provided in the recommended plan are all slide, roller, or swing gates which can be swiftly closed. These types of rapidly closing gates were incorporated as a result of previous concerns expressed by Washington level reviewers during the review of a1972 Feasibility Report for Flood Control at Buena Vista. The gates included in the feasibility study included stoplogs and other types of field assembly gates which are time consuming to install. The presently recommended street and railroad closure swing gates are designed to close by the use of a pull bar attached to the gate and the other end attached to a standard vehicular trailer hitch. The vehicle can then pull the gate shut. Any vehicle fitted with a trailer hitch can be utilized and each response team will be provided a spare bar. The roller gate at the 10th Street bridge will be closed by the use of a winch. The slide gates incorporated into the outlet structures are themselves backup gates to the flap gates and should require no additional backup capability; however, a backup source of power will be furnished as an added safety feature.

It has been determined that, with the recommended plan for flood control in place, the city of Buena Vista should respond to a standby alert status at a discharge of approximately 6,000 c.f.s. (approximate stage of 8 feet on the Maury River near Buena Vista gage). This initial response would likely be limited to inspection of the Indian Gap Run outlet structure and standing by to open the interior flood

control canal diversion gate at Indian Gap Run if the Maury River continues to rise and limits or prevents outflow through the gravity outlet. GBF personnel will also respond to a standby alert status at a discharge of approximately 6,000 c.f.s. and will be prepared to close the upstream gate on the ringwall at the northern end of the project if the Maury River continues to rise.

A review of the 51 years of systematic record indicates that an average of 2.6 alerts can be expected each year. Out of the 51 years of record, nearly 60 percent of the years would have experienced two or fewer mobilizations, while less than 10 percent of the years would have experienced more than 5 alerts. Since flood control projects similar to the recommended plan generally require annual mobilization tests, at least one of the alerts could be considered a mobilization exercise to test the locality's response to a flood event. The city is aware of the frequency of potential false alarms and has indicated that all alerts will be received positively with the appropriate response and that minimal disruptions are anticipated.

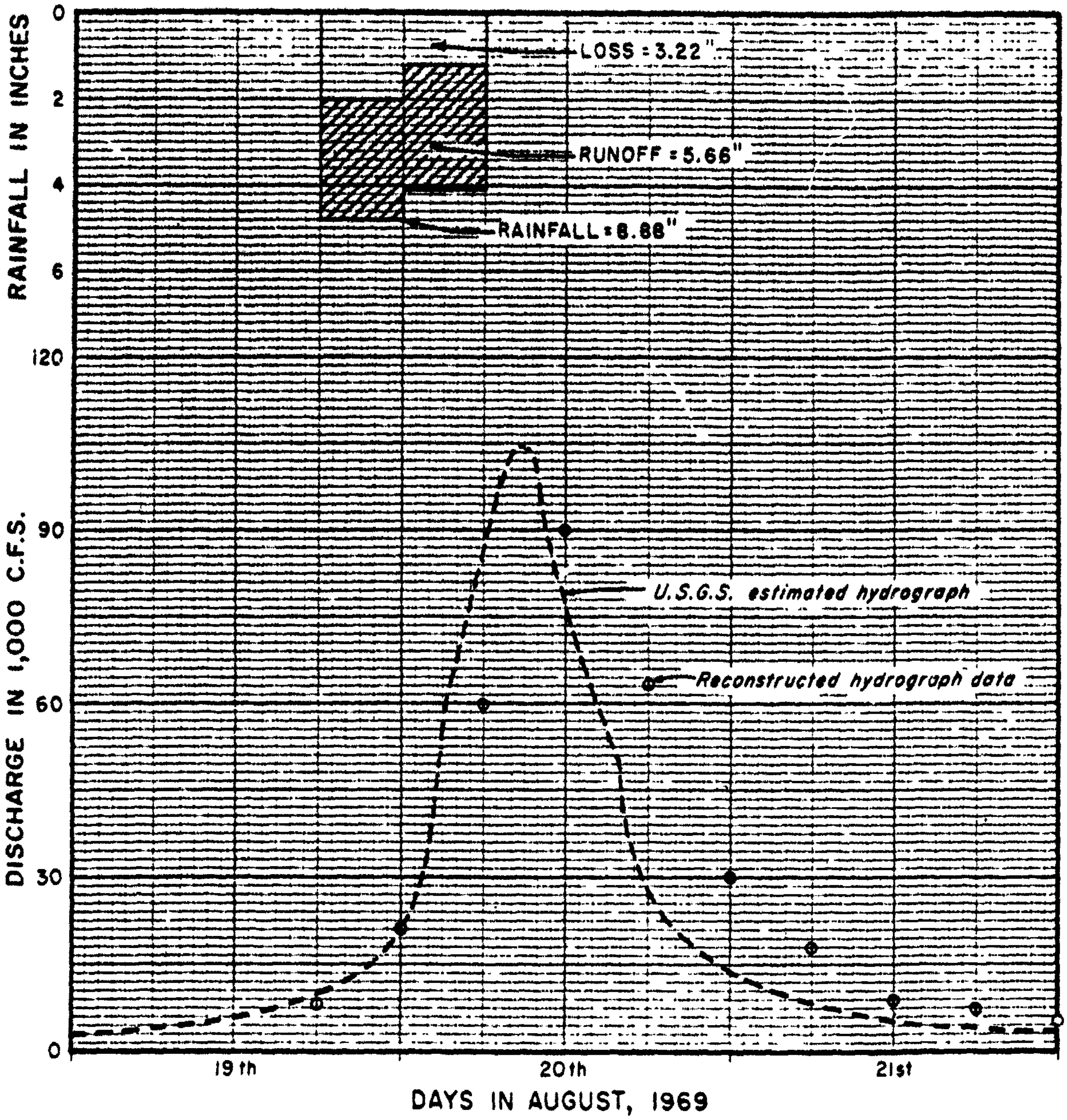
Initially, the August 1969 and November 1985 flood hydrographs (the two largest floods of record at 90,000 c.f.s., 0.87% flood event and 72,100 c.f.s., 1.4% flood event, respectively) were analyzed to determine the time available to accomplish the necessary closures. These hydrographs, shown on Figure 4 and Figure 5, respectively, are characterized by relatively slow rising limbs as a result of moderate initial rainfall. Table 2 provides a summary of the required gate operations and warning times for a reoccurrence of these two flood events. This table indicates there would be more than sufficient time (based an an alert of 6,000 c.f.s.) to accomplish the gate closures during a reoccurrence of these two flood events.

Composite Hydrograph. Realizing that a more critical rising hydrograph limb can reduce the available warning times displayed in Table 2, a composite hydrograph was developed to evaluate the workability of the various closures associated with the levee/floodwall and ringwall project. Hydrographs for the ten largest floods in the systematic record shown in Table 1 were reviewed to determine the critical rate of increase in discharge for each rising limb. These ten events represent all flood events on record exceeding 19,000 c.f.s. (approximately 25 percent flood event). Smaller flood events would not be expected to produce and maintain a more critical rate of increase in discharge.

It was determined that the September 1950 flood event produced the most critical initial rising limb above the proposed 6,000 c.f.s. alert stage. This event produced an increase in discharge from 6,300 c.f.s. to 19,900 c.f.s. in approximately 2-1/2 hours, or an increase of 5,400 c.f.s. per hour. However, this flood event peaked at only 22,800 c.f.s., well before most of the closure sills are reached by the floodwaters.

Further review indicated that the August 1969 flood event (the flood of record at Buena Vista) produced the most critical rising limb above 25,000 c.f.s. The rate of increase in discharge for this event ranged from 9,800 c.f.s. per hour to as much as 20,000 c.f.s. per hour for a short period, with an average rate of increase in discharge approximating the most critical rate of increase in discharge for the Standard Project Flood (SPF) of 13,100 c.f.s. per hour.

A composite hydrograph was then developed using the September 1950 flood hydrograph up to approximately 20,000 c.f.s. and the August 1969 flood hydrograph above 25,000 c.f.s. Between 20,000 c.f.s. and 25,000 c.f.s., it was assumed that the September 1950 flood hydrograph continued upward at the same rate of increase in discharge of 5,400 c.f.s. per hour (the most critical rate of increase in discharge for that discharge range).



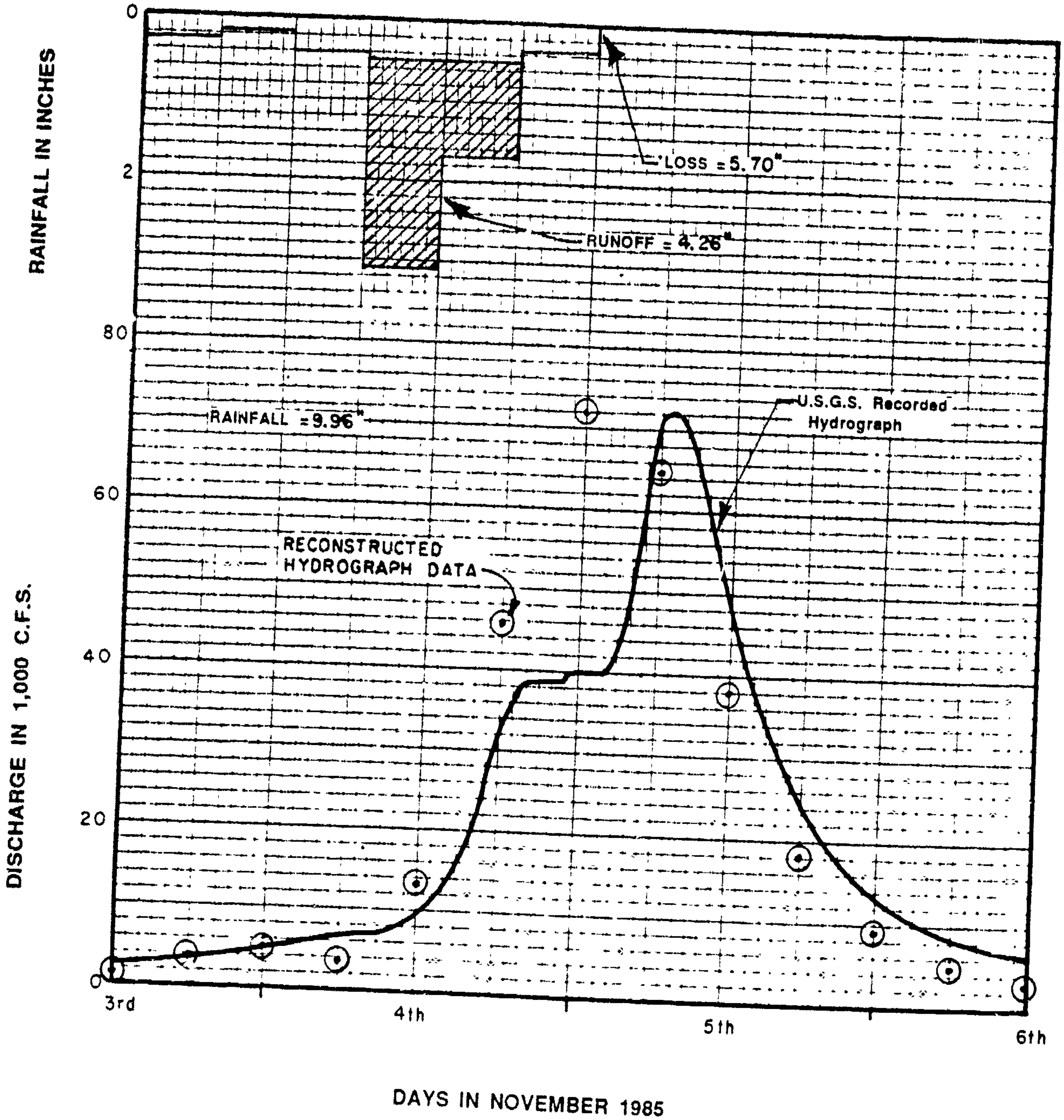
MAURY RIVER BUENA VISTA, VIRGINIA

AUGUST 1969
FLOOD HYDROGRAPH

NORFOLK DISTRICT, CORPS OF ENGINEERS

MARCH 1990

FIGURE 4



MAURY RIVER - BUENA VISTA, VIRGINIA

NOVEMBER 1985 FLOOD HYDROGRAPH

MARCH 1990

NORFOLK DISTRICT, CORPS OF ENGINEERS

FIGURE 5

Table 2. PERTINENT DATA ON CLOSURES

Closure		Time to close, hours	Actual Sill Elevation, feet, N.G.V.D.	Discharge at which closure is required, c.f s.	Exceedence frequency at sill elevation, percent	Flood risk, percent (a)	Time from flood stage to sill elevation for flood indicated, hrs. (b)	
Location	Type						August 1969	November 1985
FLAP GATES								
Indian Gap Run	Outlet - Flap	0 5	808 (c)	22,000	19	100	0 (12)	0 (12)
Diversion Canal	Outlet - Flap	0.5	816 (d)	66,000	1.8	84	3 (16)	13 (25)
OTHER GATES								
Diversion Canal	Slide	0.5	809	22,000 (e)	19 (e)	100	0 (12) (e)	0 (12) (e)
Bonded Fibers (u.s.)	Swing	0.5	844	27,000	13	100	0 (13)	0 (13)
Bonded Fibers	Swing	0 5	831	46,000	4.1	98	1(14)	10(23)
N&S RR (Hermitite)	Swing	0.5	830	56,000	2.8	94	2(15)	11(24)
N&S RR (D.S. End)	Swing	0.5	815	60,000	2.2	89	2 (15)	13 (25)
Bonded Fibers	Swing	0.5	834	63,000	2.0	87	2(15)	13(25)
Tenth St. Bridge	Roller	0.5	828	73,000	1.4	76	4 (17)	(f)
Bonded Fibers	Swing	0.5	836	74,000	1.4	76	4(17)	(f)

(a) Chance of exceeding sill elevation one or more times during the 100 year project life.
(b) Numbers in parenthesis represent warning time in hours based on a 6,000 c f s. alert.
(c) Since the flap gates operate based on differential head, the flap gates would be closed if there is no flow from the interior. invert elevation of the outlet is elevation 808, but operation is based on a river elevation of 816.
(d) Since the flap gates operate based on differential head, the flap gates would be closed if there is no flow from the interior. Invert elevation of the outlet is elevation 805, but operation is based on a river elevation of 816 at the downstream end of the levee.
(e) Same values as Indian Gap Run Closure since it will be opened at the same time the flap gates are inspected at Indian Gap Run.
(f) November 1985 flood hydrograph peaked at a discharge of 72,100 c.f s

The composite hydrograph, shown in Figure 6, was considered to provide the most critical rising limb for evaluation of the workability of the various closures. The only hydrograph to exhibit a greater prolonged rate of increase in discharge than the August 1969 flood event for discharges above 25,000 c.f.s. was the Probable Maximum Flood (PMF) with a rate of increase in discharge of 17,900 c.f.s. per hour above 45,000 c.f.s. Below 45,000 c.f.s., the August 1969 flood event had the higher rate of increase in discharge. None of the flood hydrographs evaluated (including the SPF and PMF) exhibited a greater rate of increase in discharge than the September 1950 flood event for discharges less than 25,000 c.f.s.

The composite hydrograph and the identified alert stage of 6,000 c.f.s. were then used to develop Table 3 which reflects the required mobilization (1/2 hour) and actual closure (1/4 - 1/2 hour) times and factors of safety for each closure operation. Mobilization times were based on discussions with the city as indicated earlier in this paper while closure times are based on discussions with district structural personnel. The levee/floodwall and ringwall closure analysis presented in Table 3 is considered conservative since a most critical composite hydrograph was used in the analysis and no consideration was afforded the IFLOWS system which can provide additional advance warning times based on precipitation rates and antecedent soil moisture conditions within the basin.

Effects of Nonclosure. Failure to effect closure of any of the closure structures would have varying effects on the protected areas. For instance, failure to close any of the gates at the ringwall at the northern end of the project would have no impact on the area protected by the levee/floodwall portion of the project. By the same token, failure to close any of the gates in the levee/floodwall segment of the project would have no impact on the area protected by the ringwall at the northern end of the project.

1) Ringwall. Failure to close the most upstream gate of the ringwall at the northern end of the project would have the most significant adverse impact on the area protected by the ringwall at the northern end of the project. Except within the immediate vicinity of the closure structure, velocities within the protected area would likely be no greater than would be experienced without the project. Depths of flooding within portions of the area protected by the ringwall could be greater than would be experienced without the project if the downstream street and railroad gates are shut, since the floodwaters would be entering the protected area at an elevation influenced by backwater from the existing Columbia Mills Dam, with no place to exit downstream. Ponding levels within the entire protected area could reach elevations equal to those upstream from Columbia Mills Dam. Therefore, it is recommended that, in the unlikely event the most upstream gate closure is not effected during a flood event, the downstream gates should also be left open to provide an exit for the floodwaters entering at the upstream end of the ringwall and to prevent ponding levels within the protected area from reaching depths greater than would be experienced without the project.

The potential for loss of life within the area protected by the ringwall at the northern end of the project if the upstream gate is not closed during a flood event should be no greater than without the project in place. Due to the somewhat limited area protected by the ringwall and the ready egress to high ground at U S. Route 60, the protected area can be readily evacuated before rising floodwaters reach the sill elevation of the upstream closure structure. There are no residential properties within the area protected by the ringwall which would require special advance evacuation notices. Property losses within the area protected by the ringwall at the northern end of the project would not be expected to exceed those that would be experienced without the project in place since ponding depths at most locations within the protected area would be less than without the project due to the limited opening, so long as the downstream gates are also left open.

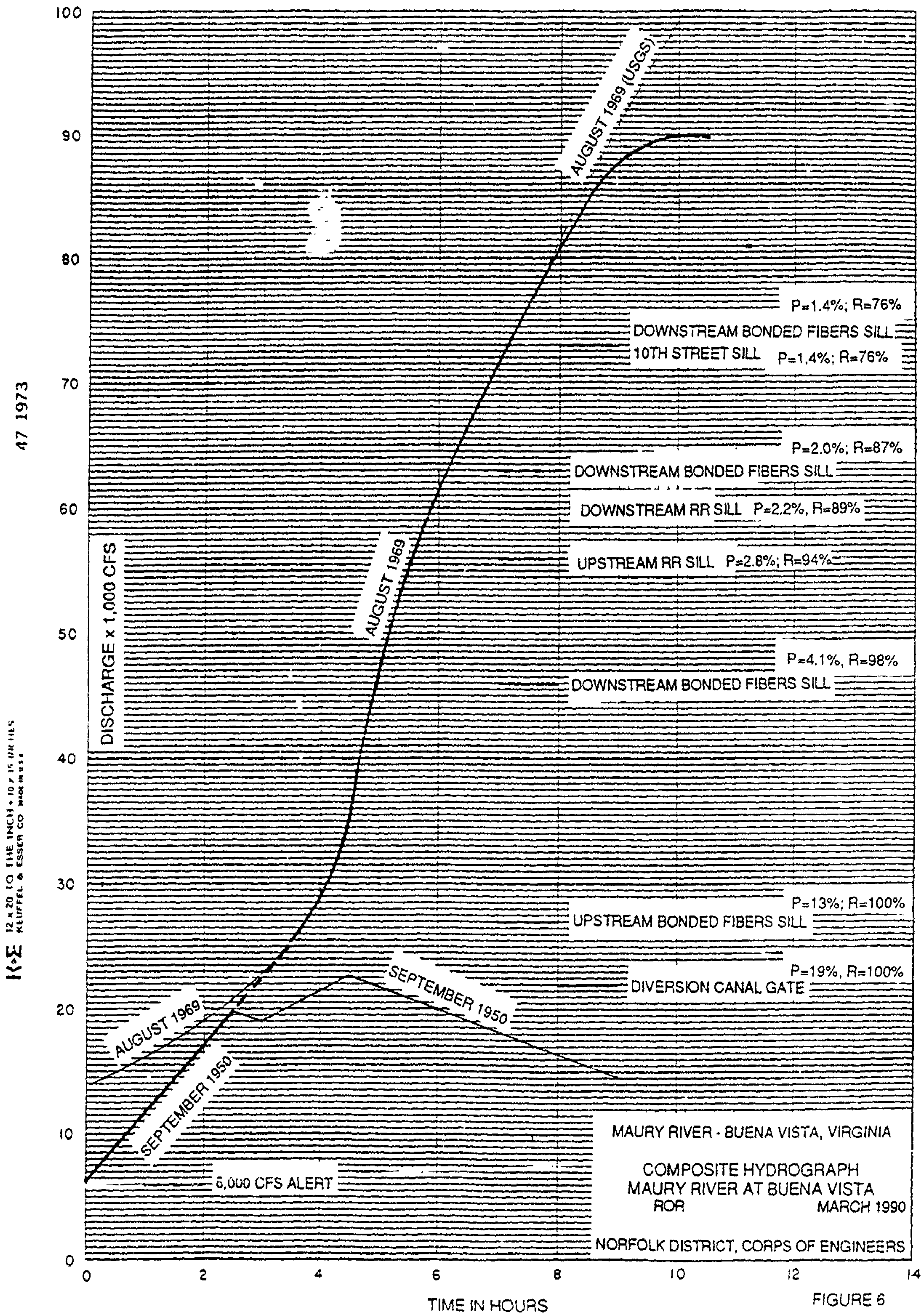

100
90
80
70
60
50
40
30
20
10
0
DISCHARGE x 1,000 CFS
0
2
4
6
8
10
12
14
TIME IN HOURS
AUGUST 1969 (USGS)
AUGUST 1969
AUGUST 1969
SEPTEMBER 1950
SEPTEMBER 1950
P=1.4%; R=76%
DOWNSTREAM BONDED FIBERS SILL
10TH STREET SILL P=1.4%; R=76%
P=2.0%; R=87%
DOWNSTREAM BONDED FIBERS SILL
DOWNSTREAM RR SILL P=2.2%, R=89%
UPSTREAM RR SILL P=2.8%; R=94%
P=4.1%, R=98%
DOWNSTREAM BONDED FIBERS SILL
P=13%; R=100%
UPSTREAM BONDED FIBERS SILL
P=19%, R=100%
DIVERSION CANAL GATE
6,000 CFS ALERT
MAURY RIVER - BUENA VISTA, VIRGINIA
COMPOSITE HYDROGRAPH
MAURY RIVER AT BUENA VISTA
ROR
MARCH 1990
NORFOLK DISTRICT, CORPS OF ENGINEERS
FIGURE 6

Table 3. SUMMARY OF CLOSURE ANALYSIS

Closure Structure Location	Type	Alert and initiation of mobilization		Sill Overtopping				Estimated closure time, hours	Available closure time, hours	Safety factor
		Q, c.f.s.	Time, hours	Sill Elevation, feet, N.G.V.D.	Q, c.f.s.	Flood Risk, percent (a)	Time, hours			
FLAP GATES										
Indian Gap Run (b)	Outlet - Flap	6,000	0	808	22,000	100	3.00	1	3.00	3.00
Diversion Canal (b)	Outlet - Flap	6,000	0	816	66,000	84	6.25	1	6.25	6.25
OTHER GATES										
Diversion Canal (c)	Slide	6,000	0	809	22,000	100	3.00	1	3.00	3.00
Bonded Fibers (u.s.)	Swing	6,000	0	844	27,000	100	3.75	1	3.75	3.75
Bonded Fibers	Swing	6,000	0	831	46,000	98	5.00	1	5.00	5.00
N&S RR (Hermitite)	Swing	6,000	0	830	56,000	94	5.50	1	5.50	5.50
N&S RR (D.S. End)	Swing	6,000	0	815	60,000	89	5.75	1	5.75	5.75
Bonded Fibers	Swing	6,000	0	834	63,000	87	6.00	1	6.00	6.00
Tenth St. Bridge	Roller	6,000	0	828	73,000	76	7.00	1	7.00	7.00
Bonded Fibers	Swing	6,000	0	836	74,000	76	7.25	1	7.25	7.25

(a) Chance of exceeding sill elevation one or more times during the 100 year project life.
(b) Operation only entails inspection of gate to insure flap gates are operating properly. This could be accomplished in advance of an alert by periodic inspections (i.e. daily, at the onset of rainfall, or some other schedule).
(c) Gate operation is independent of the line of protection (i.e. if gate operation is not accomplished, river does not enter protected area).

Failure to close any of the downstream gates of the ringwall at the northern end of the project would result in floodwaters backing into the protected area. Except possibly in the immediate vicinity of the closure structure, velocities within the protected area would be significantly less than would be experienced without the project in place. Ponding levels, and associated property losses, for virtually the entire area protected by the ringwall would also be less than would be experienced without the project. The potential for loss of life for this scenario is minimal since floodwaters would back into the protected area at relatively low velocities and, as discussed earlier, the limited area protected by the ringwall can be rapidly evacuated to high ground at U.S. Route 60.

2) Levee/Floodwall. Failure to effect closure of the most upstream gate of the levee/floodwall portion of the project (the railroad closure near Hermitto) would have no impact on the protected interior area until a flood event slightly greater than the March 1936 flood (approximately equal to a 3% flood event which has a 95% chance of being exceeded one or more times during the 100 year project life) is experienced. For flood events approaching the design flood level, there could be some minor limited flooding as the floodwaters entering at the upstream end of the protected area flow downstream toward the gravity outlet structure at the downstream end of the diversion canal. Depending on the magnitude and timing of any associated interior rainfall/runoff, the gravity outlet structure at the downstream end of the diversion canal would likely have sufficient capacity to pass the entering floodwaters with little or no significant additional ponding and associated damages. Except in the immediate vicinity of the closure structure, velocities and depths of flow for this scenario would likely be less than would be experienced without the project.

Failure to effect closure of the 10th Street Bridge would have similar, but even less potential adverse impacts than those associated with nonclosure of the upstream railroad closure structure. For flood events up to the magnitude of the November 1985 flood (approximately equal to a 1.4% flood event which has a 76% chance of being exceeded one or more times during the 100 year project life), failure to effect closure of the 10th Street Bridge would have no impact on the with project interior flood levels within the protected area of the levee/floodwall portion of the project since a reoccurrence of the November 1985 flood would peak just below the sill elevation of the closure structure at the 10th Street Bridge.

The two gravity outlet structures are fitted with flap gaps to prevent intrusion of floodwaters from the exterior into the interior protected area. So long as the flap gates are operating properly and no debris has jammed the gates open, there would be no adverse impact on the interior area if the gate is not inspected for proper operation during a rising flood event. In the unlikely event that the flap gates at the Indian Gap Run outlet structure are not inspected and one or more of the flap gates are not operating properly, floodwaters would back into the protected area. Ponding levels, and associated property losses, within the area protected by the levee/floodwall would be less than would be experienced without the project as the entering floodwaters would flow downstream toward the gravity outlet structure at the downstream end of the diversion canal. The potential for loss of life for this scenario is minimal since floodwaters would back into the protected area at relatively low velocities If the flap gates at the outlet structure at the downstream end of the diversion canal are also not inspected and one or more of the flap gates do not operate properly, floodwaters would back into the protected area. However, due to the influence of the downstream channelization and levee extension, interior ponding levels, and associated property losses, would be minimal. The potential for loss of life for this scenario is also considered minimal since floodwaters would back into the protected area from the downstream end of the project at relatively low velocities.

Failure to effect closure of the downstream railroad closure of the levee/floodwall portion of the project would have minimal impacts on the interior area due to the effects of the downstream channelization and levee extension. For flood events up to and including the March 1936 flood (approximately equal to a 3% flood event which has a 95% chance of being exceeded

one or more times during the 100 year project life), failure to effect closure of the downstream railroad closure would have no impact on the interior flood levels with the levee/floodwall in place since a reoccurrence of the March 1936 flood would peak somewhat below the sill elevation of the closure structure. For larger flood events, the potential for loss of life is significantly less than would be experienced without the project since floodwaters would back into the protected area as opposed to flowing through the protected area. In addition, interior ponding levels, and associated property losses would be minimal due to the influence of the downstream channelization and levee extension.

Overtopping. Since the design flood for virtually every levee or floodwall local protection project is not the maximum flood that can occur, there is an inherent risk that the levee or floodwall will be overtopped during the life of the project. There is approximately a 58 percent chance of exceeding the recommended level of protection (0.87% flood event) at Buena Vista during the 100 year project life. Overtopping of a previously protected area can, if ignored, significantly increase the chances for loss of life. A separate analysis outside of the scope of this paper was accomplished to ensure that overtopping of the recommended plan at Buena Vista initiates in the least hazardous areas. The city of Buena Vista has indicated they are aware of the potential for overtopping and are willing to accept the associated risks. The following discussion identifies the actions the city of Buena Vista must accomplish to minimize the chances for loss of life within the protected areas if overtopping of the line of protection is expected.

Portions of the institutional arrangements discussed earlier in this paper will be utilized to monitor flood conditions and, when necessary, issue evacuation notices. When it appears that a potential for overtopping exists during a rising flood event, the city's EOC will notify the public safety personnel, including the police and volunteer fire and rescue squads, to initiate evacuation procedures. The city's Police Department consists of 15 full-time paid personnel, while the volunteer fire department has 25 active members and the rescue squad has 37 active members. These personnel are sufficient to carry out the evacuation procedures for the areas protected by the levee/floodwall and ringwall portions of the project. The areas that would require evacuation include 6 major industries, approximately 85 commercial structures, and approximately 185 residences.

Since the Maury River can be a relatively fast rising stream, the intent of any evacuation notice due to a potential for overtopping of the line of protection is limited solely to protection of life and limb. Without a significant number of false alarms, sufficient advance warning of impending overtopping cannot be provided in order to minimize loss of property by relocating personal possessions and other moveable contents out of the flood threatened area. On this premise, the areas that would be impacted by overtopping of the line of protection can be evacuated in 1 to 1-1/2 hours from the time the decision is made to evacuate. Sufficient evacuation routes are available along U.S. Route 60 east, Route 501 in either direction, and a number of local streets that feed into the higher residential areas within the city.

When an evacuation order is issued, the emergency personnel will give emergency notice on a door-to-door basis. If conditions dictate otherwise, the evacuation notice will be given by public address system from emergency vehicles and by radio and television. Evacuation shelters have already been identified at local schools that are located outside of both the floodplain and the areas protected by the levee/floodwall and ringwall. During the passage of Hurricane Hugo in September 1989, these emergency personnel were utilized to serve advance door-to-door notices to heighten awareness and prepare for a possible later evacuation order. In that particular instance, an evacuation order was not necessary.

To evaluate the effectiveness of the evacuation plan, a rate of increase in discharge of 13,100 c.f.s. per hour was adopted. As discussed earlier in this paper, this rate of increase in discharge compares to a sustained rate of discharge increase for the SPF and August 1969 flood hydrographs. At a discharge of approximately 30,000 c.f.s. (approximately equal to a 10% flood event which has approximately a 100% chance of being exceeded one or more times during the 100 year project life), emergency personnel will be alerted and initial evacuation preparatory notices issued. This represents a minimum notice of 6 hours in advance of overtopping. Since all of the emergency personnel reside within the city limits, it will require less than 1/2 hour for the emergency personnel to respond and begin issuing initial evacuation

preparatory notices. During the 51 year systematic record, such initial notice would have been issued only three times, in 1936, 1969, and 1985. If the Maury River continues to rise, actual evacuation will be ordered at a discharge of approximately 57,000 c.f.s. (approximately equal to a 2.7% flood event which has approximately a 94% chance of being exceeded one or more times during the 100 year project life). This provides a minimum of 4 hours to complete evacuation, or a safety factor of 2.7 to 4.0 based on a 1 to 1-1/2 hour evacuation period. Actual evacuation would have been ordered only twice during the 51 year systematic record and in neither instance would the line of protection been overtopped.

The levee/floodwall and the ringwall are both designed for overtopping to initiate on the downstream ends to reduce velocities within the protected area by creating a backwater type flooding and to reduce the impacts associated with sudden overtopping by providing an initial cushion of water on the interior areas. However, since the Maury River can be a relatively fast rising stream, the project design cannot ensure that sudden overtopping will not occur for flood events significantly larger than the design flood. Therefore, it is recognized that the interior areas can fill in a relatively short period and velocities within the protected areas can be significant. However, proper execution of the evacuation plan will minimize the potential for loss of life within the protected areas during a flood event large enough to produce overtopping.

The duration of flooding within the protected area following an overtopping event is not expected to be significantly longer than would be experienced without the project. As the flood hydrograph that produced overtopping begins to recede, flooding on the interior area protected by the levee/floodwall will fall to approximately elevation 827 feet, NGVD, the elevation corresponding to the top of the levee at the downstream end of the project. As the flood hydrograph continues to recede, the 3-8'x8' flap gate equipped gravity outlets at the closure structure at the downstream end of the diversion canal will begin discharging the ponded floodwaters based on a differential head. Once the river stages opposite the Indian Gap Run gravity outlet falls below the elevation of the floodwaters trapped behind the line of protection, the ponded floodwaters will also begin discharging based on a differential head through the 5-10'x10' flap gate equipped gravity outlets. These two gravity outlet structures have sufficient capacity to allow the interior pond elevation to follow the receding river stages.

Flooding on the interior area protected by the ringwall as a result of overtopping will fall to approximately elevation 842 feet, NGVD, the elevation corresponding to the downstream portion of the ringwall, as the flood hydrograph that produced overtopping begins to recede. As the flood hydrograph continues to recede, a differential head will develop between the interior and the unprotected side of the ringwall. One or more of the downstream vehicular access gate or railroad gates will be designed to open if a significant differential head on the interior develops. Once the downstream closure structure(s) opens, the interior ponded floodwaters will quickly reach the elevation of the river due to the somewhat limited volume of floodwaters stored within the protected area. The elevation of the interior pond will continue to drop as the river recedes.

Conclusion

Development of a critical composite hydrograph to evaluate the workability of the closure gates and evacuation plans confirmed that the levee/floodwall and ringwall plan is a workable and viable alternative. However, the composite hydrograph does point out that the warning times available for the Maury River at Buena Vista can be much shorter than the warning times actually experienced during the two largest floods in the period of record. Therefore, it is critical to the functionality of the project that the local sponsor continuously monitor weather and river conditions to ensure that adequate warning times are provided at all times. In addition, it is critical to project operations and implementation of evacuation plans that operations and emergency personnel respond when notified. It is imperative that the locals do not become complacent as a result of inactivity or false alarms.

OPERATIONAL HYDROLOGY
SOME PLANNING CONSIDERATIONS

by

John Burns[1]

As most of you know, our report preparation and approval process has been modified to improve our responsiveness to our non-Federal partners. Pursuant to WRDA '86 our planning studies are now completed in two phases.

The reconnaissance phase commences with the issuance of appropriated reconnaissance funds requested by an FOA, and terminates with the execution of a feasibility cost sharing agreement or the division commander's public notice for a report recommending no Federal action. Studies undertaken during the reconnaissance phase are conducted at full Federal expense, exclusive of any costs incurred by non-Federal interests in volunteered work or services during the phase. The reconnaissance studies should accomplish four very important tasks.

(1) In conjunction with the potential non-Federal sponsor, identify the problems and the array of potential solutions to these problems.

(2) Determine that there is at least one potential solution that will likely have a Federal interest, will be in accord with current policies and budgetary priorities, and will be supported by the non-Federal sponsor.

(3) An estimate of the time and costs for the feasibility studies.

(4) A draft feasibility cost sharing agreement and a letter of intent from the potential non-Federal sponsor stating that the report is acceptable and that the agreement will be signed upon certification of the report.

The reconnaissance study will normally be completed within 12 months and by law cannot exceed 18 months. The cost of the reconnaissance studies is limited to 25 percent of the total estimated cost of the reconnaissance and feasibility studies.

Upon completion of the reconnaissance study a reconnaissance review conference (RRC), involving the local sponsor and all corps echelons, will be scheduled. The purpose of the RRC is to insure that the report is consistent with current policies and budgetary priorities. The RRC occurs prior to release of the report to the public and prior to division approval of the reconnaissance report and FCSA. The conclusions of the RRC are documented in a memorandum of the meeting and a followup HQUSACE response.

The HQUSACE certification process for the reconnaissance report, the negotiated FCSA, and the letter of intent follow the RRC. Certification is designed to ensure that proceeding to the feasibility phase will maximize the chance that a project will be implemented. Certification constitutes HQUSACE approval of the reconnaissance report, the negotiated FCSA, and the sponsors letter of intent to execute the FCSA.

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After certification, the remaining activities in the reconnaissance phase are: release of the reconnaissance report to the public: execution of the FCSA; and HQUSACE release of funds to initiate the feasibility phase. Normally these actions can be accomplished within a month after certification.

During the reconnaissance phase engineering involvement is needed to develop alternatives, define engineering efforts required for the feasibility phase, develop the preliminary cost estimate and schedule, and provide support for negotiating the FCSA. It is essential that all members of the study team participate in this important scoping process. Detailed engineering is generally not required during the reconnaissance phase. Assessment should be based on minimal analysis and should rely strongly on engineering judgement. A technical review conference should be held at the end of the reconnaissance phase to reach consensus on the engineering aspects of the FCSA (i.e. Agreement on the Total Engineering effort and products to be performed during the feasibility phase to obtain a good baseline estimate) and on the need for subsequent technical review conferences and engineering documentation during PED.

The feasibility phase starts with the issuance of initial Federal feasibility funds, following execution of the feasibility cost sharing agreement, and terminates on the date the feasibility report is submitted to OMB by the Assistant Secretary of the Army for Civil Works for review for consistency with the policies and programs of the President.

The purpose of the feasibility study is to ensure the timely and economical completion of a quality feasibility report that is expected to recommend an implementable solution to the identified water resources problems. the report should be a complete decision document, that presents the results of both study phases. In addition to providing a complete presentation of study results and findings, the feasibility report should indicate compliance with applicable statutes, executive orders, and policies and provide a sound basis for decision makers to judge the recommended solutions.

The cost of the feasibility phase is shared equally between the Federal government and the non-Federal sponsors and is normally completed within 24-36 months. At least 50% of the non-federal sponsor's share must be in cash. The remainder of the non-Federal sponsor's share, up to 25 percent of the total feasibility phase cost, may be in-kind products and services.

A feasibility review conference (FRC) is held prior to release of the draft report for public review. The purpose of the FRC is to seek Washington level commitment to the project in order to minimize the potential for significant modification of the remaining studies and the final feasibility report recommendation after the final report is submitted for Washington level review.

HQUSACE will prepare a project guidance memorandum (PGM) in response to the meeting MFR. The PGM will contain the requirements each recommended project must meet to be supported by HQUSACE and ASA(CW). Further Washington level review of feasibility reports beyond the FRC will take place only to the extent required to determine how the final report responds to issues raised at the FRC, to guidance provided, and to State and agency comments.

The focus of engineering during the feasibility study is on establishment of project features and elements, design assumptions, assessment of available data, and collection of new data necessary to prepare an accurate baseline cost estimate for the project which is presented to Congress for authorization.

Detailed engineering studies and analyses should be scoped to the level needed to establish project features and elements that will form an adequate basis for the project construction schedule and

baseline cost estimate. Uncertainties should be reflected in contingencies which will be resolved during PED.

Once a report is finalized and the division commander has issued his public notice the report is forwarded to Washington for subsequent processing to the Congress. We have revised our Washington level review procedures to reduce the time it takes to get a report to Congress while retaining the decisionmaking prerogatives of each echelon.

We have streamlined our review process, developed a management system to bring our studies and projects in on time and within budget, and in general have become much more aware of the need to provide an acceptable, workable and sound product by the end of the feasibility phase.

During the feasibility phase we strive to develop a project, acceptable to our non-Federal partner with a solid cost estimate, that is consistent with our overall program for Federal water projects. To that end, it is important that the operation and maintenance requirements that our non-Federal partner will face be clearly articulated and that the necessary analysis be completed to insure that the project can be operated and maintained as planned.

During the planning stages it is particularly important that we analyze and evaluate the expected performance of a project in operation. We are particularly concerned that the project we are planning can actually be operated as it is intended. It is vitally important that the energies of all members of the study team be brought to bear on this question. It is only through the cooperative efforts of the entire team questions concerning the operation of the project can be addressed.

As a particular case in point, I would like to examine some operational considerations that have come up on some recent flood control studies.

For projects that include levees consideration should be given to the area between the levees. In particular, will the project perform as expected assuming typical controls on development or do we need to incorporate special provisions in the Local Cooperation Agreement for the project to operate as expected.

A second consideration deals with the ability of the local sponsor to operate the project as expected. Does the sponsor control the necessary resources and have the necessary expertise to operate the project? Can the project physically be operated as expected? The combined efforts of the study team members should be brought to bear on analyzing such factors as the sponsors capability to operate the project, training needs, and physical impediments to carrying out items such as making closures.

As part of the project formulation process, the study team will formulate the National Economic Development (NED) Plan and assess the tradeoff between first-cost and O&M. In making this tradeoff the study team should be sure that the plan that is finally recommended will operate as designed. Realistic estimated of O&M are needed to insure that low first-cost/high O&M projects are evaluated on the same basis with other potential projects.

In summary, it is vitally important that all members of the study team be involved early and that they gain a good understanding of how the project is intended to function. Each member of the study team will be more effective if he or she has a good understanding of all aspects of the project and can provide the study team with a clear picture of any "stovepipe concerns" that might arise as the potential project moves through the process towards construction.

CORPS OF ENGINEERS
Hydroelectric Power
(Current Status & Policy Review)

by
S. A. Zanganeh[1]

INTRODUCTION

The purpose of this paper is to discuss the following issues relative to hydroelectric power development:

- National perspective

  a. Current status
  b. Physical potential
  c. Demand for electricity
  d. National energy strategy

- Hydropower In Corps of Engineers

  a. General
  b. Authority and policy
     (1) Federal development
     (2) Non-Federal cost shared development

NATIONAL PERSPECTIVE

a. Current Status - The amount of hydroelectric power generated varies from year to year, depending of course on the availability of water. Total generating capacity of hydropower nationwide is currently about 72,000 megawatts - which represent 10 percent of the nation's total generating capacity. In addition, there is about 17,000 megawatts of pumped storage capacity in the U.S. Hydroelectric power continues to be the most economical source of electricity, averaging about 0.7 cents per kilowatt hour. Federal projects account for 44 percent of current hydropower capacity. It should be noted that the Administration policy on the use of Federal funds, as it has been recently expressed by OMB is that: Federal funds may be used for the study of addition to or expansion of hydropower facilities at new multiple - purpose dams. The use of Federal funds for the study of new single-purpose hydropower projects is not permitted. The OMB statement is consistent of course with the current Corps policy that allows the use of Federal funds for reconnaissance and feasibility studies; however, the use of Federal funds for feasibility studies, based on current Corps policy is limited to 50 percent of the total cost, the balance of it will have to be financed by non-Federal entities who will be project sponsors.

b. Physical Potential - The physical potential for expanding hydroelectric power resources is high. A FERC report indicates that in U.S. we have developed about 50 percent of our hydropower potential. However, environmental, statutory, and regulatory constraints can add to the cost of

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expansion projects and substantially reduce estimates of development potential. The best sites are at existing dams and range up to 24,000 MW (FERC Data). In addition 16,000 MW could be achieved from upgrading the existing hydroelectric projects.

c. Demand for Electricity - Not only we do have the potential for development, we also have substantial demand for electricity which is increasing faster than it was expected a few years ago. North American Reliability Council's 10-year projection indicates that the U. S. by year 2000 needs about 210,000 MW of new capacity to meet the demand. This projection accounts for plant retirements and 20 percent reserve capacity. Electric utilities are making plans to provide electricity into the next century. All various supply options are being investigated to determine how best to meet the expected growth in demand for electricity.

d. Hydropower a Renewable Resource - The largest contribution from renewable energy sources today comes from hydropower. Hydropower provides 46 percent of U. S. renewable energy supply. Hydropower competes with conventional fossil capacity in base load, intermediate, and peaking roles. To place the current U.S. hydropower capacity in perspective, the equivalent of some 150 coal-fired plants are displaced by hydroelectric resources. Hydropower brings substantial reliability and cost effectiveness to the electric generation system.

e. National Energy Strategy - On July 26, 1989, the President directed the Secretary of Energy to initiate the development of a Comprehensive National Energy Strategy. The DOE report will be submitted to the President in December 1990. The Corps is one of the agencies providing data on hydroelectric power to DOE for use in the renewable energy segment of the National Energy Strategy Report. In participating at some of the working group sessions, I see that there is tremendous efforts to address all aspects of hydroelectric power from the regulatory complexity to the number of laws and decisions affecting hydropower development. An interim report on National Energy Strategy was made available by DOE on April 1990 for review and comment. I believe that focus on renewable energy and conservation will make a come back.

<u>Hydropower In Corps of Engineers</u>

a. General - The generation of hydroelectric power at the Corps of Engineers projects is one of the important public benefits derived from development of the Nation's water resources. Beginning with the Bonneville project on the Columbia River in 1938, the Corps as of this year (1990) have in operation 74 multiple purpose projects with nameplate capacity of 21,000 megawatts. This is the largest block of hydroelectric capacity constructed by a single agency in our country, and it represents about 30 percent of the Nation's hydroelectric generation. In addition, non-Federal development of hydroelectric power at existing Corps dams under the provisions of the Federal Power Act is continuing to increase, we now have 50 constructed non-Federal power plants at Corps projects with a total capacity of about 1600 Megawatts. We also have 16 power plants under construction by non-Federal entities and about 36 plants in design and approval process for construction at existing dams. In FY 1989, the cost of the Corps technical review of non-Federal hydropower proposals amounted to $1.3 million of which $803,063 was reimbursed

to the Corps directly by non-Federal entities and the balance of the expenditure ($469,063) was reported to FERC for collection. These statistics show that the number of federally financed and constructed projects (74) has not increased over the past several years; however, the number of add-on hydropower plants has gone up from 21 to 50. We can expect that add-on power plants by non-Federal entities will continue to dominate our program. Such expansion is supported by the potential for development that exists at Corps dams. A recent IWR publication shows that there are 146 Corps projects with no hydropower development.

b. Authority and Policy:

(1) Federal Development - Congress has authorized the Corps to study hydropower development as part of multiple purpose water resources development. Therefore, hydropower has always been an incidental rather than a primary purpose of water resources development by the Corps.

(2) The formulation and evaluation of Federal hydropower development, until October 1973, was based on the requirements of Senate Document No. 97, 87th Congress, 29 May 1962. On 25 October 1973, new principles and standards became effective through the Water Resources Council (WRC) under the authorities provided in the Water Resources Planning Act of 1965. The Water Resources Development Act of 1974 modified these principles and standards with respect to the applicable interest rate. Basically, the approach for evaluation of Federal hydroelectric projects in WRC standards is the same as that of original Senate Document No. 97 and it requires briefly the following considerations:

(a) The power should be usable in and adaptable to the requirements of the overall regional power load (the FERC is consulted on this feature).

(b) The total project benefits should equal or exceed total project costs.

(c) Power benefits must equal or exceed the separable cost for including power in the project. The usual practice is to measure the benefit in terms of the cost of achieving the same result by the most likely alternative means that would exist in the absence of the hydro project. The alternative is usually a privately financed thermal power plant.

(d) Comparability Test: The separable hydropower costs should be less than the cost of the most likely alternative means of providing equivalent service in the absence of the project, evaluated on a basis of taxes, interest, and other financial factors comparable with the determination of project costs (Federal financing).

(e) Financial feasibility: Insofar as can be determined in advance, potential net revenues should be sufficient to repay power costs. Costs allocated to power should be recoverable with interest within a reasonable period. Administratively, the repayment period has been set at 50 years. Financial feasibility of a project should be determined by requesting the marketing agency for their views as to the revenue to be expected from the sale of the power that would be generated by the proposed project.

It should be noted that Principles and Guidelines allows a marketability study to be substituted for the "need for power" analysis for small projects having an installed capacity of 25 MW or less. This means that for 25 MW or less we don't have to do extensive economic analysis for various plant sizes. All we need is a letter from a power marketing agency that there is a "need for power" within the project marketing area.

The question arises if we are going to have any Federally financed hydropower project. The answer is <u>not</u> likely. I realize that some of our documents such as the "Policy Digest" indicates that when non-Federal hydro development is not practical, "impractical" Federal Development can be pursued. Frankly, I have never personally accepted the concept of impracticability, I don't believe it is a question of impracticability, the real question is if we want to-accept an added burden in water control management. An example of impracticability is where we don't want existing Federal power plants to be expanded by non-Federal entities, because of operational difficulties, such as two entities operating the same project. We now obviously have learned that we can solve this type of problem as evidenced by recently approved cost-shared add-on hydropower projects.

At the Fort Gibson project in SWD, two 1[illegible] MW units will be added to the existing Corps powerhouse by the Grand River Authority. In addition there is a proposal in SWD for upgrading the units at the Dardanelle Lock and Dam powerhouse (current installed capacity is 4-units of 124 MW) by partnership arrangement with a non-Federal entity who will be willing to pay the entire cost of the upgrade, provided he receive the added increase in capacity (about 16 MW) resulting from the upgrade, usually 10-15 percent of the original capacity. MRD also has received similar proposals. The OCE position is that in this type of arrangement if non-Federals commit to 100 percent of the costs in a signed Memorandum of Agreement (MOA) with the Corps, the upgrading of the unit(s) could go ahead. Note that the Corps will continue to operate the project's power plant, and the power marketing agency will be marketing the additional power for the private entity. As I see it, currently the only impracticability is when we can not find a sponsor to pay for either adding units or upgrading the existing units.

With the above policy and the fact the Corps' powerhouses have 134 units that are <u>over</u> 30 years old, and another 80 units that are 20-30 year old, we can see the potential need, not only for add-on hydropower but also for upgrading, is substantial. I suspect that a good portion of this type of work will be done by non-Federal financing.

b. Non-Federal Cost-shared Hydropower Development - From the above discussion non-Federal financing is the current policy with which we want to develop, expand, or improve hydropower resources at the Corps projects. The following presents a summary of procedures in formulating a partnership project:

(1) Consistent with Congressional authority in connection with multiple purpose studies, such as a 216, etc., we still have to address the potential of hydropower installation at our project under study; however,

recommendation for development (feasibility study, design, and construction) should include a discussion of potential sponsor(s) who are willing to cost-share 50-50 the feasibility study and commit funds for 100 percent of construction, and operation costs allocated to power.

(2) We can conduct, at Federal cost, reconnaissance study for establishing the potential of hydropower and identifying a sponsor to cost-share 50-50 on the feasibility study. This type of study could be for add-on power at existing projects without power (where there is no proposals for development through the FERC), expansion of number of units at Corps powerhouses or upgrading of units at existing Corps projects. A MOA for commitment of funds for the feasibility study and payment for full costs of design and construction is necessary.

(3) Hydropower development at existing dams are also accomplished, as you know, through the licensing procedures of FERC. Currently, design and construction of add-on hydro at Corps projects through FERC licensing is the largest segment of Corps hydropower program. As I stated earlier I expect this type of development will continue to be a major part of our program in this decade. In talking about add-on hydropower, we should also count the substantial number of relicensing applications that we have began to receive from FERC for review and approval. These are non-Federal plants either at the Corps dams or at private dams that are up for upgrading not only for additional power, but also for meeting the current design and environmental requirements for a new license.

References:

- Water Power & Dam Construction, August 1989
- Directory of Corps Dams with Existing Hydro- electric Power Facilities and/or the Potential for addition of Hydroelectric Power, July 1988, IWR
- The Potential of Renewable Energy, An Inter-laboratory White Paper, DOE, March 1990
- Interim Report, National Energy Strategy, DOE, April 1990

THE ROLE OF OPERATIONAL HYDROLOGY IN ADDRESSING CORPS' WATER CONTROL ISSUES

by

Douglas D. Speers[1]

INTRODUCTION

Throughout the Corps of Engineers today water control managers are faced with an ever-increasing number of challenges to defend existing reservoir operating procedures and regulation policies and to develop regulating strategies that address new public concerns. In North Pacific Division alone, reallocation of reservoir storage is being studied for several reservoir projects; long-standing procedures for regulating reservoirs have been analyzed and revised; and, the privatization of one Corps multiple-purpose reservoir is soon to be consummated.

More than ever before operational hydrology is being called upon, sometimes in new and unique applications, in the field of water control management to address such problems. This paper describes some examples of such applications in North Pacific Division, and summarizes needs and concerns relating to the application of operational hydrology in dealing with problems and issues in water control.

EXAMPLES OF WATER CONTROL ISSUES AND ASSOCIATED HYDROLOGIC INVESTIGATIONS

Columbia River Flood Control Rule Curves. The Northwest Power Act of 1980 brought into existence a new regional agency in the Pacific Northwest that has oversight responsibilities for power planning and fisheries resource management in the Columbia River basin. Answerable to the governors of the Northwest states, the Northwest Power Planning Council established as one of its first actions a Fish and Wildlife Program, which tasked water management agencies with designated responsibilities to help preserve a dwindling salmon fishery. One of the tasks for the Corps of Engineers was to examine its flood control rule curves to see if advantages could be gained to provide additional stored water for the fishery. In response to the request, the North Pacific Division office embarked in 1984 upon a 5-year study of flood control criteria for several major multiple-purpose storage projects in the basin. Three reports were produced, and rule curves for several projects were modified and are now being used in operations. The guidelines established by the Corps under which the study was conducted were as follows:

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1) The rule curve investigation was to be no more than a technical re-evaluation of existing operating criteria, with no evaluation of storage reallocation to be considered.

2) Any changes in rule curves were not to worsen the existing degree of protection, as defined by standard project flood levels and stage-frequency relationships at downstream control points.

Following these criteria, the study resulted in modified rule curves such as that shown in Figure 1 for Libby project. The capability of long-range forecasting, because the majority of runoff is from snowmelt, permits usage of variable flood control rule curves shown. The study tended to verify the original curves for high forecasted runoff volumes, but indicated that the original curves were too conservative for low runoff years. The changes were generally favorable to conservation storage operations (and the fishery interests) in that they tended to remove an operational restriction that enhanced reservoir refill and the filling of storage for instream usage.

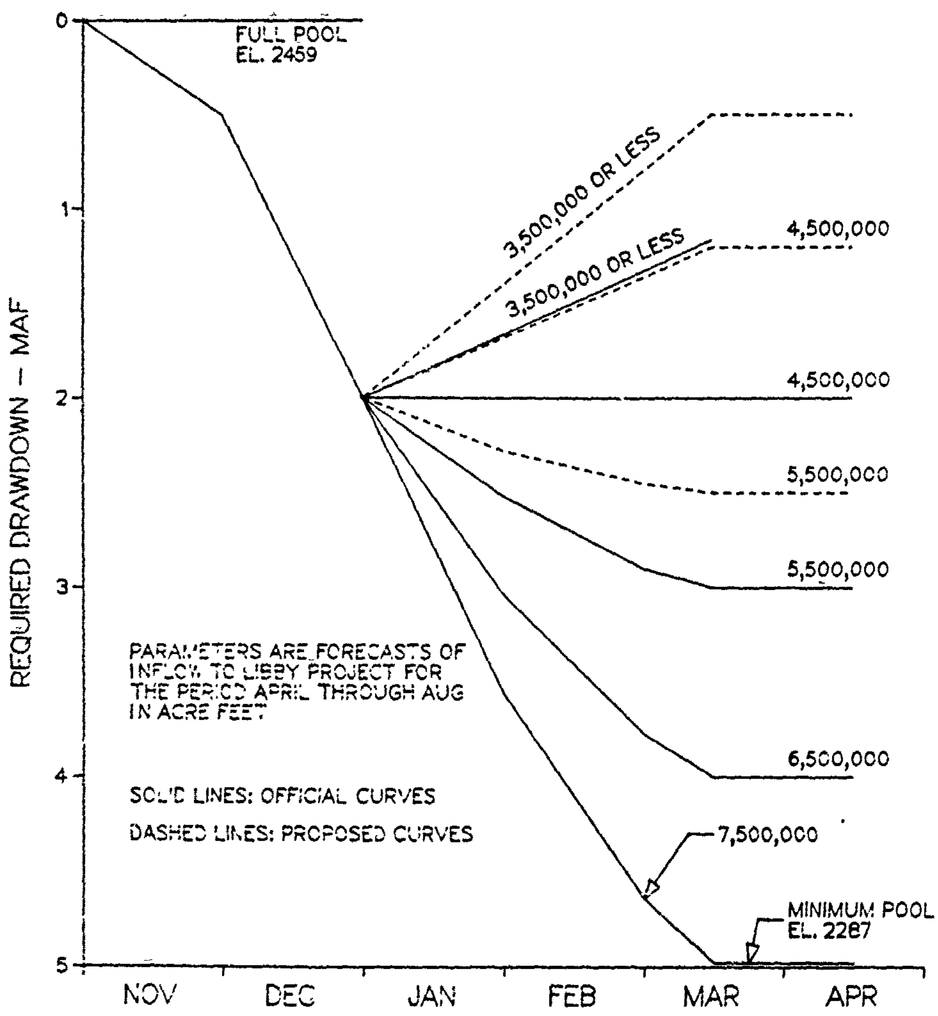

Figure 1. Original and Modified Flood Control Curves, Libby Project

The study made extensive use of hydrologic modeling that is described in the 1988 Hydrology Workshop proceedings (Speers, 1988). For the NPD office, this represented the most complex and large-scale analysis that it had ever undertaken for the basin. The basic objective was to evaluate the effects of unforecastable spring rainfall given various combinations of snow conditions, storage levels, and rainfall patterns. This involved calibrating new watershed models, deriving synthetic spring rainfall patterns, and performing a large number of alternative simulations that included watershed, river and reservoir operations simulation. The SSARR hydrologic model was used for this study.

The study was completed in 1989 and a draft version of the final report was furnished agencies for their comment. Comments received indicated a somewhat reluctant acceptance of the changes (greater benefits of the study had been hoped for by some agencies), and a questioning by some of the basic criteria described above. These questions may have to be addressed again in the System Operation Study described below.

System Operation Review Study. The North Pacific Division office, in cooperation with the Bonneville Power Administration and the Bureau of Reclamation, is embarking on the region's single largest study to the Columbia River Basin, called the System Operation Review Study (SOR). This study, which is estimated to cost upwards of $10 million and take 4 years to complete, is brought about by the necessity to renew key multi-agency operating agreements governing the planning and operation of the hydroelectric system. Since future agreements on the operation of the system will have significant effects on the environment, agencies will employ strict NEPA procedures which will involve a full scale Environmental Impact Statement on the operation of the system. It is expected that many operating issues will be surfaced in the public involvement process and will require study. A combination of GI and O&M monies will finance this project.

Extensive hydrologic/reservoir system studies will be required for the SOR, involving all three agencies. One backbone computer model that will be used by the NPD office is the HYSSR program, which simulates the seasonal operation of the system on bi-weekly time steps. Hourly simulation models, production-cost models, and hydrologic models will also be employed.

Willamette River Review Study. The Portland District has underway a study of the its 13 reservoirs in the Willamette Basin. Authorized and funded through specific legislation, the study is brought about largely by the very strong recreation interests that have developed in the Basin. Here the issue is one of flood control versus reservoir refill. The study authorizes a reexamination of the operating criteria for the reservoirs in the system, as well as the investigation of some potential new developments that would enhance refill. The study was initiated in 1989 and is scheduled for completion in 1991.

Hydrologic analysis for this study will primarily utilize the HEC-5 program to assess alternatives of seasonal flood control rule curves, with the goal to evaluate the potential for enhancing summer refill in the interest of lake recreation.

NEEDS AND CONCERNS

Experience in NPD with the cases described above reveal the following needs and concerns with respect to operational hydrology applications for water control:

Public Scrutiny of Study. Experience with the cases described above revealed clearly that hydrology studies of existing water control facilities can be expected to receive a great deal of interest and close scrutiny by affected agencies and local interests. Furthermore, it is quite possible that this may reach the political arena. In the Columbia River Flood Control Study, this interest was established early in the study and continued in the form of questions and requests for briefings as the study progressed. This scrutiny applied to even the most technical of details, which are normally not contested in the planning, engineering, and design phases of new projects.

Study Methodology and Scope. Because of the public attention given to operational hydrology studies, the methodology and scope of the study need to be carefully considered to assure credibility of the results and acceptance by the affected interests. Several potential problems exist:

1) Specific concerns by local interests, technically well founded or not, have the potential for driving the study beyond the scope and complexity that might be considered desirable from the Corps' perspective.

2) Affected agencies and local interests, naively influenced by a limited knowledge of "state of the art" technology, may question Corps' standard hydrologic engineering methodologies. This may require an allocation of resources to explaining and defending the procedures chosen.

3) The study scope deemed to be required from the technical point of view may be so extensive that time schedules and resources are strained. This presents internal problems of justifying the study to management and obtaining resources.

Guidance and Technical Tools. Experience in North Pacific Division indicated no serious limitations with technical tools to utilize in carrying out operational studies, nor lack of guidance in interpreting results. In the latter case, although the policy involved may be relatively straight-forward and clear (e.g., a modification of rule curves is not to worsen downstream degree of protection), the public and political interest involved may necessitate HQUSACE participation. Regarding technical tools, the Corps' library of models and other computer programs, along with technical consultation from HEC and other labs, seems adequate to accommodate virtually any operational study. However, NPD experience is that skill and ingenuity is challenged in applying these tools, perhaps more so than in planning and design.

Funding. Given the need for careful and perhaps extensive hydrologic/operational analysis, funding of large studies becomes a problem. Fortunately, in the case studies described above funding was available through both continuing and special authorizations. Perhaps the greater problem is maintaining the capability to undertake even small-scale operational studies in this era of extremely limited O&M funds. This requires careful and proactive measures by water control managers in identifying the concern, explaining and "selling" the need for the study to program managers, and working within the O&M budget process - through stovepipe support - in getting funds established in the budget.

REFERENCE

Speers, D.D., "Calibrating and Applying a Model of the Columbia River Basin," _Proceedings of a Workshop on Calibration and Application of Hydrologic Models_, Gulf Shores, Alabama, October, 1988.

HYDROLOGIC ENGINEERING FOR EFFECTIVE WATER CONTROL MANAGEMENT

by

Richard J. DiBuono[1]

1. Introduction.

a. **Purpose.** Hydrologic engineering during the feasibility analysis and design or reformulation of water resources development projects that involve hydraulic structures must include consideration of their ultimate operability to achieve the intended objectives. Communication of the water control management factors important to the project plan is often not sufficiently treated during documentation for the review process. The intent of this paper is to indicate why communication of the water control aspects of such projects is important to the decision process.

b. **Key Issues.** Future monetary costs and personnel resources that will be needed to operate a water control facility are often overlooked or understated in the project authorization process. Now that the law requires local sponsors of all new water projects, including reservoirs, to be responsible for operation and maintenance, it is imperative that owners understand the resources that must be committed for successful operation. Similarly, communication of the objectives and benefits of a water control plan for an existing project is important in the public acceptance process. Increasingly, the public and elected officials are scrutinizing our water control policies and practices, and often seek understanding in economic terms. Gone are the days when the Corps of Engineers and the Congress alone determined a project's fate.

2. Discussion.

a. **Determining Operability.** From time to time, project reports are transmitted to Washington for review and approval that adequately explain the hydrologic engineering basis for their size and configuration but information needed to determine if in fact the project could be operated as assumed in the static design condition is lacking. Water control management considerations such as the number and distribution of precipitation and streamflow gages or other means needed to acquire operational data and the availability of a system to transmit and manage the data in real-time may be essential to the project's feasibility. Consideration must be given to the response time needed between onset of a storm and the proper

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setting or closure of gates or other such water control facilities if the design is to yield a functional project. Further, such water control factors must be explained in the project documentation.

In one recent case, a coastal flood protection project was seriously questioned about its operational feasibility by the Washington Level Review Center during its review for the Board of Engineers for Rivers and Harbors. A gated tidal barrier across the mouth of a river, its successful regulation during major storm tides would require determining when during the tide cycle to close the barrier so that the storage volume in the estuary behind it would be sufficient to safely contain the coincident interior river runoff. The feasibility report adequately demonstrated that capability using historic flood data, but failed to communicate the practicality of acquiring data and making water control management decisions on a real-time basis during future events. The means for collection and management of the necessary weather, streamflow and tide data, and the professional expertise of the sponsor's personnel were subsequently determined to be sufficient to prove the project's feasibility. Discussion of these matters in the initial feasibility report would have prevented delay in the approval process.

b. **Managing Existing Projects.** Although it has long been the practice in the formulation phase of water project development to coordinate the efforts of the hydrologic engineer and the economist (though, it must be said, not always to a mutually satisfactory degree), such joint involvement of the two specialties has not generally occurred during the operations phase. Now, however, the situation is changing. Impetus for change comes from the desire of the public and elected officials to understand the basis for our project water control plans and for the occasional decision to exercise the Secretary of the Army's discretionary authority to deviate from a plan. In order to develop acceptable plans and gain support for our management decisions, we need to better communicate the effectiveness of our policies and practices in economic as well as hydrological terms. Situations that have focused recent attention on our water control management activities range from droughts to floods.

1) **Drought Situation.** The extensive droughts in the southeast, the Mississippi River basin, and the far West during the past ten years have focused the attention of the general public, special interest groups, and, therefore, government leaders on the manner in which the Corps both develops and executes its reservoir water control plans. During times of water shortage, the various purposes served by the projects often are competing and in some cases are directly conflicting. Many questions are raised by the interested and affected parties as to how priorities are established among the various project purposes and what bases exist in either legal or in economic terms. The

legal questions usually can be readily answered by reference to specific project authorizing legislation or generic laws, such as the Clean Water Act, Fish and Wildlife Coordination Act, etc. However, we find it much more difficult to answer the questions pertaining to the monetary value accruing to the purposes a project serves. Though the Corps collects and amasses large hydrologic data bases upon which to base its water control decisions, the collection of economic data for an operational project usually ceases after the formulation phase.

Nowhere in the Corps has this situation been brought into sharper focus than in the current drought management of the main stem Missouri River reservoir system. Four consecutive years of severe drought have resulted in the lowest reservoir levels since the system was first filled in 1967. A virtual conflict has developed between the upper basin states, where the lakes are located and where lake-based activities are reported to have significant importance to their economy, and the lower basin states where flow augmentation of the Missouri River by releases from the reservoirs serve many of the economically important purposes included in the system's authorization. Because we have not been able to prove to the satisfaction of all parties that our system water control plan is equitable and in consonance with its authorization, the Missouri River Division has embarked on a major study to update and review the existing Master Water Control Manual. Central to this investigation will be the extensive collection of economic data which will be used together with the existing hydrologic data base to develop and apply a prescriptive systems operation model. While some believe that the study may result in proving the adequacy of the current plan, this is the magnitude of the effort that has been required because we were unable to explain its economic viability.

2) **Flood Situation.** Post-flood analysis of damages prevented by the operation of reservoirs and other water control facilities requires knowledge of the relationship between river stage and the value of property at the index sites along the affected reach of the river downstream. All too often, the stage-damage relationships have not been carefully reevaluated since the project's formulation phase. An additional complexity in the post-flood study is determination of the proper allocation of benefits between projects when, for example, the flood protection was provided by a combination of upstream reservoirs and a local protection project such as a levee or floodwall. These are further examples of the need to coordinate the work of the hydrologic engineer and the economist in communicating the effectiveness of our water control management activities. Unfortunately, funds to collect economic data are not customarily made available from the O&M General Account. However, such data are valuable for planning investigations so sources of General Investigation Account funds should be explored.

c. **Assisting Policy-makers.** At times effective communication of hydrologic and hydraulic engineering information may prove critical to formulating a decision by the Assistant Secretary of the Army (Civil Works) on an issue of major regional or national significance. Such was the case in the navigation crisis during the drought-impacted summer of 1988 when Mississippi River flows fell to record low levels. The Assistant Secretary was asked by the Governor of Illinois and other elected officials from Mississippi River basin states to exercise his authorities to increase the amount of water diversion from Lake Michigan to augment the flow in the Mississippi River. The proposal was vehemently opposed by the Great Lakes basin states and the government of Canada. Analysis by the Corps of Engineers determined that the maximum amount of additional flow that could be safely diverted would indeed raise the river stage at St. Louis by as much as one foot but that, because of its unstable alluvial channel, there was no certainty that there would be a corresponding increase in depth for navigation. Using this information, the Assistant Secretary was able to make a decision that was understandable to parties on both sides of the issue and saved him from having to make a decision that would have the appearance of favoring one group over the other.

3. **Conclusions.**

Effective water control management of Corps projects starts with consideration of their operational feasibility during the formulation phase. Sponsors must know the costs and resources needed for water control over a project's lifetime before committing to sharing costs for design and construction. Communication of the effectiveness and benefits of our water control plans is essential for gaining public acceptance and Congressional support. To achieve this, documentation and reporting of the water control management aspects of projects must explain issues from the economic as well as the hydrologic perspective.

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